Ground deformation from the Christchurch Earthquakes

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ABSTRACT

Three case studies are presented to show how significant ground deformation occurred with the Canterbury earthquake sequence, which in turn resulted in slopes to shallow foundations. The case studies are all from the north-western part of the Christchurch CDB where a shallow gravel layer overlies sand, some of which is liquefiable. Deformations as derived either from pre- and post-earthquake topographic surveys or from building floor levels show 250 – 300mm differential over 100m. This is likely to be a result of a combination of liquefaction at depth, seismic consolidation of looser soils, and the inevitable deformation from irregular seismic waves combined with differences in soil strength in all three dimensions. Although these mechanisms can explain the deformation, analysis tools as commonly used are approximate at best. The out of levelness as recorded at the case sites may well have little impact on building functionality, but in the post-earthquake climate, this has tended to be lost in an attempt to maximise financial recompense to the detriment of the recovery and the environment. Ultimately earthquake engineering is about reducing the impact of events on people and society. Part of societal resilience is mental and attitudinal. Engineers have a role to communicate the possibility of ground and hence foundation deformation, as part of building a more resilient infrastructure and society.

1 INTRODUCTION

The origins of this paper lie partly in observations of ground deformation resulting from the Canterbury Earthquake Sequence of 2010 - 2011, and partly in how engineers and the wider society have responded to out of level buildings. The first part of this paper presents evidence of ground deformation. There are numerous papers which in one form or another point to the complexity of liquefaction and the limitations of the simplified methods of liquefaction analysis (Crawford et al 2019, Cubrinovski et al 2017, Pyke 2020, Pyke & North 2019). This paper does not address analysis beyond showing how the methods commonly used in New Zealand do not provide estimates of ground movement that correlate well with observation. The case study examples chosen are all from the Christchurch CBD, on similar soil profiles where shallow liquefaction and sand ejecta was very limited, and hence soil volume loss and foundation movement from shallow underlying liquefaction can be essentially eliminated as significant contributors. . In this, they also show how the resulting building deformation is not "foundation" settlement but is better regarded for what it is: as ground settlement. This may seem an unimportant distinction, but is significant in how the building performance is perceived.

The second part briefly discusses what this means for building foundation performance and the implications for communicating this to society. The issue has also become entangled by engineers adopting advocatory roles and simply poor judgement in the insurance arguments following the earthquakes. It concludes with the author's views of how foundation deformation could be better framed in a world increasingly beset with environmental imperatives.

2 GROUND DEFORMATION IN PART OF CHRISTCHURCH CBD

Part of central Christchurch is underlain with a relatively consistent soil profile. The area between the Botanic Gardens and to east of Colombo St and between a little north of Armagh St to south of Tuam St is underlain with sandy gravel from about 2 – 3m depth to about 8 – 10m depth, overlying predominantly sand to 18 – 20m where a silty zone, often with some organics, caps the Riccarton gravel at 21 – 23m depth. The area is shown indicatively in Figure 1. Groundwater table is typically at 2m to 3m depth. There was little recognizable ground damage in the form of liquefaction ejecta or ground cracking in this area with the 2010-11 earthquakes.

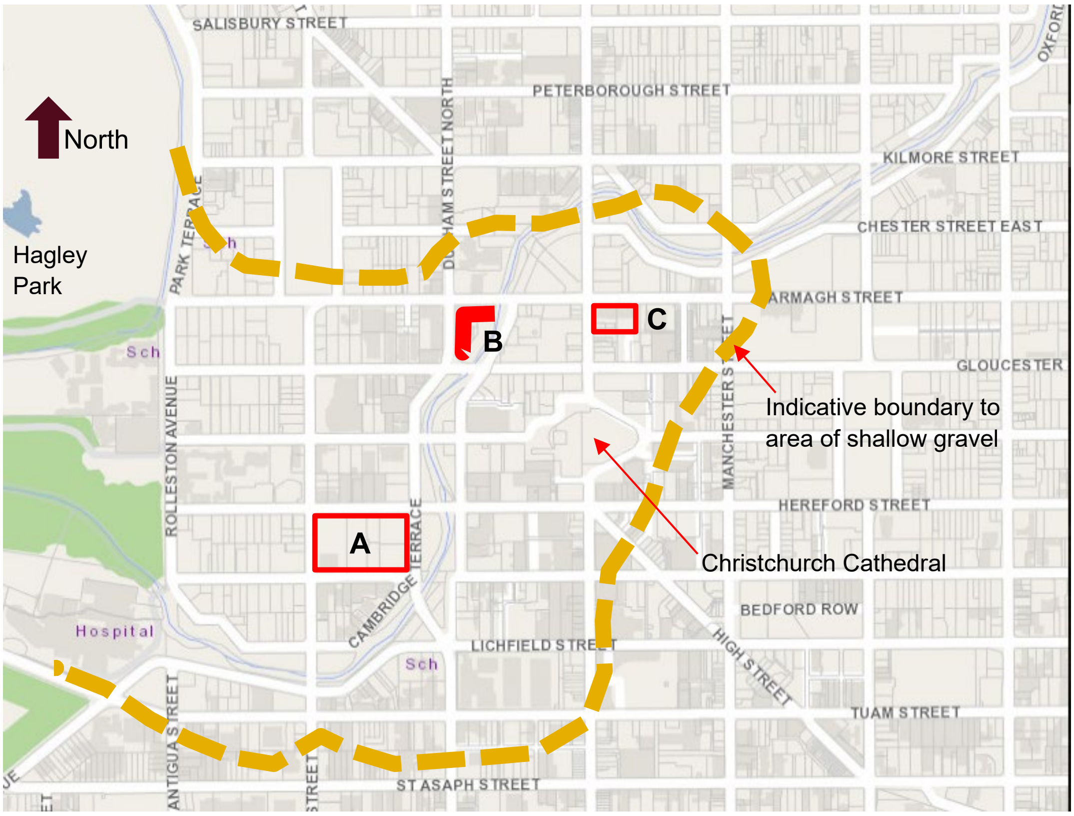
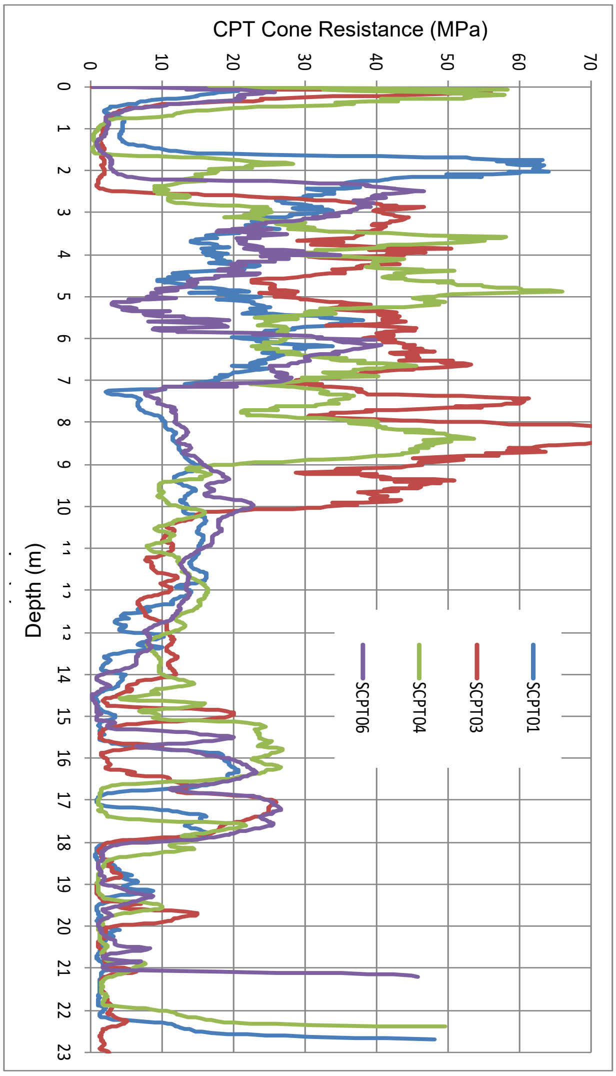


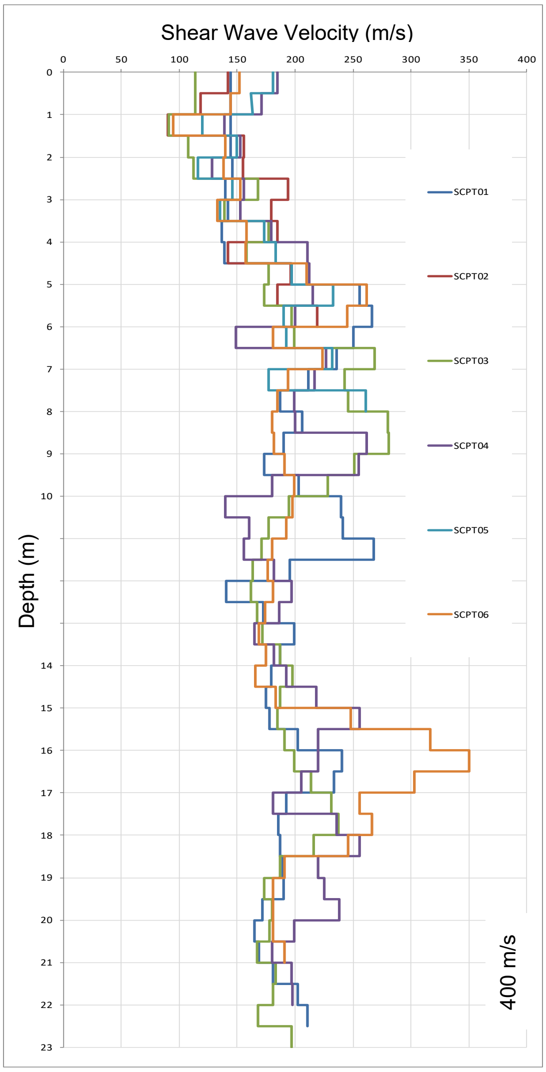
Figure 1: Map of central Christchurch showing location of case study sites

Three sites provide an interesting insight into surface deformation. Liquefaction susceptibility has been assessed for each site using the simplified cyclic stress method as set out in Boulanger & Idriss 2014 with post-liquefaction settlement by Zhang et al 2002. A fines correction factor of 0.07 was used for three sites, together with a 15% probability of liquefaction. As a higher CFC value is likely to be more realistic (Leeves et al 2015), as would a higher probability when comparing with observed settlements, the analysis is likely to be overpredicting the extent of liquefaction. This is in addition to recent reporting that more sophisticated methods of analysis give significantly lower estimates of liquefaction and settlement. Thus the difference between 2-dimensional liquefaction analysis prediction and observed settlement is likely to be greater than the comparisons below suggest. The level data available for the sites dates from after the earthquake sequence and thus represents the cumulative effects of all the earthquakes. The analysis has included the four major events: M7.2 September 2010, M6.2 February 2010, M6 June 2011 and M5.9 December 2011. The vertical post-liquefaction consolidation settlements thus estimated suggest that the analysis is a poor prediction tool at these locations. In addition soil-structure interactions do not appear to be predominant drivers for these three sites.

2.1 City block south of Hereford Street

The author was responsible for geotechnical advice for this city block, measuring 100m by 160m in plan (marked A in Figure 1). Most of this block was empty of buildings through the earthquake period. There was liquefaction ejecta only at the eastern end of this block. The CPT and shear wave profiles derived from a seismic cone for four tests to the Riccarton Gravel are shown in Figure 2 and show the near surface looser sand over the denser gravel to between 7 and 10m depth over sand interbedded with some looser and siltier layers at depth with the Riccarton gravel contacted at between 21m and 23m depth. The overall pattern of this profile is typical for the wider area, including the other two sites referred to below.





(a) CPT Profiles *(b) Shear Wave Velocity Profiles*

Figure 2: Test Data for Hereford Street Block

Topographic level data for the complete city block from 2006 was able to be compared with a similar survey from 2015. Assuming kerb levels were originally laid to consistent grades, it was possible to make reasonable interpolations between the two surveys. Assuming that the datum for both surveys was unaffected by the earthquakes, the kerb settled by about 0.35m in both northeast and northwest street corners, with a little less between, and decreasing to about 0.1m at the southwest corner and perhaps as little as 0.05m at the southeast corner. Even if the datums were not comparable, the north side of the block settled 0.25m to 0.3m more than the south side. This is generally consistent with the pattern in the LiDAR for all events. Although analysis indicated some shallow liquefaction was likely in about 40% of the 29 CPT tests on the site, the predicted shallow liquefaction induced settlement was less than 30mm at all the CPTs for the Feb 2011 EQ. CPT testing was constrained with the shallow gravel often limiting penetration depth, but on this site five tests were able to be made full depth to the Riccarton gravel. Analysis of these tests showed that liquefaction induced settlement of looser sands below the gravel were greater at 25 – 150mm. Cumulative liquefaction settlements for all four major EQs were calculated and showed poor correlation to actual settlement, with predicted values ranging between 1 (correct prediction) and 2.8 times the measured values, or suggest the differential of about 250 to 300mm over a distance of 100m. There is a slightly better correlation with average SPT values from five deep boreholes, showing a trend of decreasing settlement with increasing average SPT , which suggests that seismic consolidation in the gravels is contributing to the vertical ground deformation.

2.2 Durham – Armagh St

A second site is the Canterbury Provincial Buildings on the corner of Armagh and Durham streets (B on Fig.1). The original parts built in 1859 – 61 are largely of timber, but with a three storey stone tower on Armagh St, a stone safe on Durham St, and several transverse masonry fire walls. The large stone Council Chamber was built at the south end in 1865 and a stone section added to the east end at a later date.

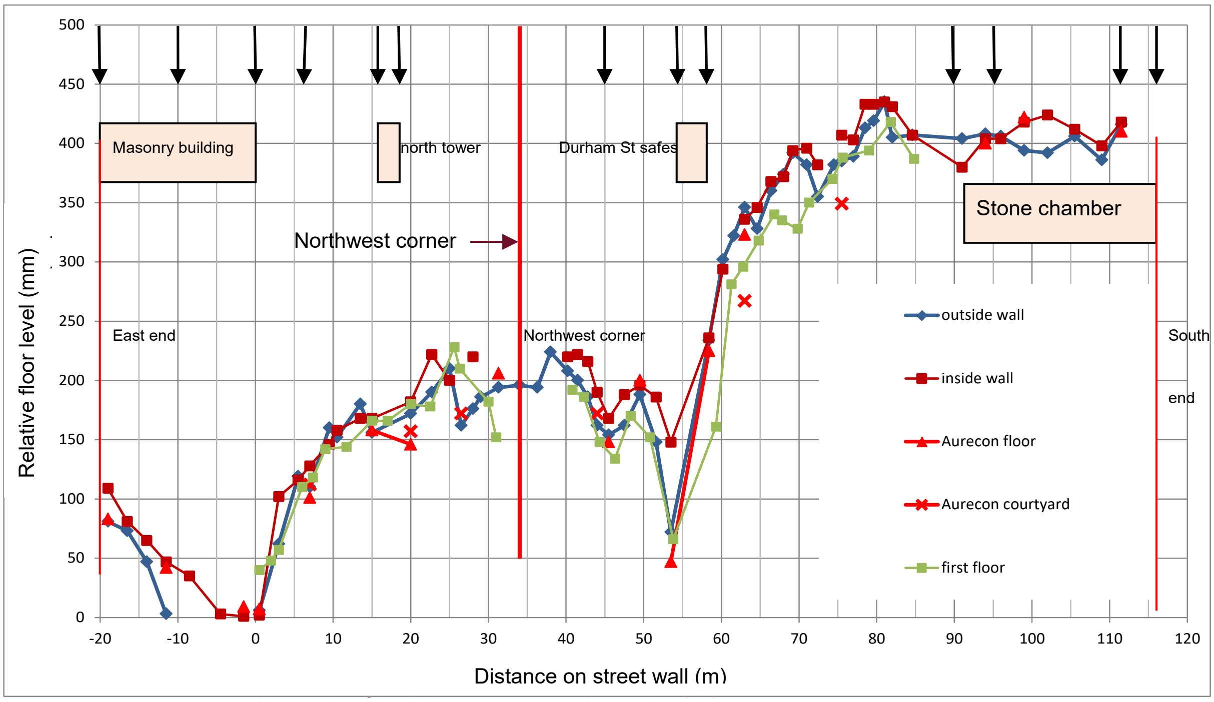
Figure 3 Canterbury provincial buildings in nineteenth century. The stone Council Chamber is far right

Several level surveys after the earthquakes are plotted in Figure 4, along the approximately 140m length of the L-shaped building.

The building shows 450mm difference in floor levels along the building length. The south end, including the heavy stone chamber, is relatively level, as is a section to each side of the building corner. The most severe differential settlement occurs on the south side of the Durham St safes with about 160mm settlement over about 5m and at the east end of the timber section on Armagh St with about 100mm over 3m (both about

1:30). Most of the level differences is within a 90m length to give an overall slope of about 1 in 200. Some floor settlement is evident with slopes down to many, but not all, of the masonry cross walls. The masonry building at the east end slopes the opposite way to the overall trend with a slope of about 1:120. There is very limited evidence of settlement of the foundation relative to the adjacent ground.

Estimation of settlement under static conditions suggests less than 25mm for the masonry sections and less than 10mm for the timber sections.



Arrows at top indicate position of transverse masonry fire walls

Figure 4: Relative floor levels on longitudinal section on Armagh & Durham St buildings

There was very limited liquefaction ejecta on or close to the site, in any of the earthquakes, although liquefaction is predicted in the sands below the shallow gravel. The shallow gravel extends to above the water table under most of the site, and shallow liquefaction should be very limited in extent. The shallow gravel also extends under the Avon River bed, and lateral spread is not perceived as being a significant hazard. There was limited ground cracking of which the most marked was perpendicular to the river.

Liquefaction is predicted in the looser sand below the shallow gravel, but estimates of settlement (Zhang et al 2002) for the February 2011 earthquake ranged from 40 to 100mm, with the larger value from a CPT at the south end, which is least affected by settlement. The liquefaction analysis does not predict the level differential in either magnitude (observed differential is about four times that predicted) or in the overall slope from south to north/ east.

2.3 Armagh St

The third site is further east on Armagh St (C on Fig. 1). The 18 storey building is built on a 24m square raft. The surface sand was removed and replaced with compacted hardfill in the northwest corner where the shallow gravel dipped below the founding level and thus the entire foundation rests on gravel. This building was monitored during construction in 1988, and the raft settled 23mm over the first 6 months during the time when virtually all the dead load of the building was being placed, and an additional 3mm over the following

2 months. Total settlement was probably less than 30mm, with a tilt on the diagonal towards the northwest of 8mm. After the earthquake, the tilt had increase to about 70mm, still to the northwest.

At about 10m to the east of the raft is a row of columns on shallow 2.5m square pads supporting a podium. Post earthquake levels on the first floor beam soffits indicate 50 – 60mm differential level to the north. The next door building along was two storey on shallow strip foundations. The end wall, about 5m from the podium pads also showed a level differential of 70 – 90mm to the north.

There are therefore three foundations of raft, pads and strip, all carrying different loads, yet all showing a consistent trend in tilt to the north (or northwest) of similar magnitude. This is also consistent with the trend shown in LiDAR levels in the adjacent streets. Given the similarity in differential settlement of three different foundations (raft, pads & strip) under three different structures (18 level tower, 3 storey podium, 2 storey), it is difficult to see how soil structure interaction in the normal sense could be a significant component of the tilt.

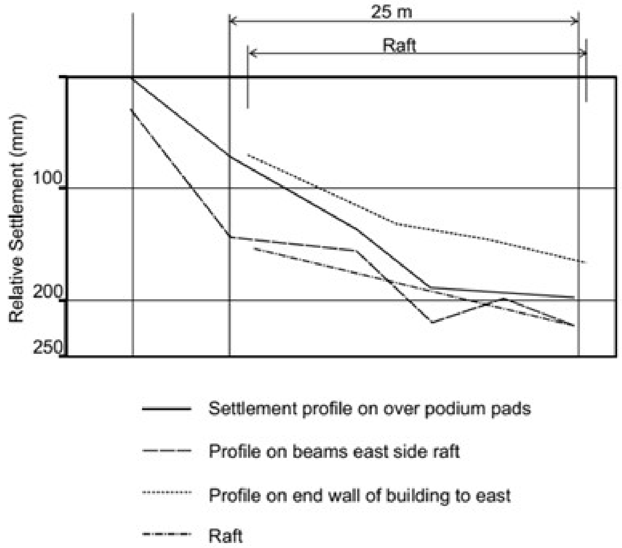


Figure 5: Relative settlement profiles of three adjacent Foundations, post CES

Deep CPT data for this site included one test immediately on the south side and three tests about 50m from the site to the northwest and northeast. Liquefaction induced settlement estimates for the February 2011 EQ were 110mm for the one to the south and 150 – 180mm for those to the north. Thus the recorded building tilts could be in response to the liquefaction at depth.

2.4 Summary of Observations and Discussion

The three sites all show vertical ground deformations. The city block had few buildings on it at the time, with limited liquefaction ejecta at one end only and the observed settlements are derived from street levels remote from foundations. The Provincial Chambers level profile bears little correspondence to the foundation loads; the heaviest part of the complex, the stone chamber at the south end, is now the highest part of the building, and the Armagh St site has three very different foundations with very different loadings following a similar pattern.

All the sites are on a similar soil profile, which at face value would not be expected to deform to this degree. There has been a tendency to consider only liquefaction induced settlement for Christchurch buildings, but there is clearly more going on. The upper 20m of soils in Christchurch are geologically very young, and although they would have experienced shaking form distant earthquakes, may well not have been subject to the very strong shaking experienced in the February 2011 earthquake. As well as liquefaction, consolidation of looser but not liquefied soils should be expected throughout the soil profile. With the variable soils present in Christchurch, differences in soil strengths will lead to different shear strains and hence different deformations. The differential ground movements observed are likely to be the sum of the liquefaction, consolidation, seismic wave distortion and deformation from variable ground.

As the variability is 3-dimensional, these different deformations will occur both horizontally and vertically. Although the above case studies are focused on vertical deformation, it is also noted that there was obvious horizontal deformation along Armagh Street, with a series of pavement cracks observed west of the Provincial buildings, a compression bulge between Colombo St and the Avon River, a tension fracture across the tram tracks outside the third site, and the tram track, acting as a long relatively incompressible beam, punching into the outside of the bend at New Regent Street to the east. There was some ground cracking in the Provincial Buildings site, both parallel to the Avon River, suggestive of some deformation towards the river bank, but also perpendicular to it.

The differential settlement across the city block in Hereford Street is not easily explained, but both the Provincial Buildings and the Armagh Street sites are towards the northern edge of the shallow gravel and some of the movement at these sites is likely to be from deformation in, and towards, the softer soils to the north. With a view considerably wider than the specific site itself, greater ground deformation could perhaps be predicted in principle, even if not by routine numerical analysis. While the potential for differential ground movement can be predicted, the magnitude cannot, at least with any precision. Even where there is standard liquefaction settlement analysis available, the results are often very approximate.

3 IMPLICATIONS AND EXPECTATIONS

As demonstrated above, the earthquake shaking propagating through the deep soils under Christchurch resulted in ground deformations which in turn produced differential movement in buildings. A building taking half the width of The Hereford St block would have settlement differentially 100 – 150mm with 1:300 – 1:400 slope. The Provincial Buildings have over 400mm differential settlement with average slopes of 1:200. For a new building on these sites subject to a similar earthquake in the future, would they require revelling, and if so, how could this be done economically? Or is some slope acceptable? For building foundations, functionality is closely related to deformation, in fact, foundation failure can be (and often is) defined in terms of excessive deformation. Some deformation can and does occur without impacting on functionality (see MBIE Guidance 2012 & 2014 for background on this). As the overall floor slopes are not excessive for most building uses (indoor bowling or printing presses excepted), does it matter? In the post-2011 environment where many insurance policies included the delightfully ill-defined term “as good as new”, it did matter. Next time, New Zealand may not be so lucky. In addition, there has been an understandable human response of doing it better for next time, resulting in performance specifications for some buildings which this author doubts are actually achievable on ground such as Christchurch.

Deep ground improvement methods would certainly reduce the extent of differential ground movement, but unless extended to close to 25m depth, can not be expected to eliminate it. Similarly, even deep piles are likely to show some movement as they are subjected to seismic loading exceeding the normal static case, plus stress redistribution from shaft to tip, plus the potential for some soil consolidation below the tips (the Riccarton gravel, although much older than the soils above, is not bedrock, and still variable in density and grading). The improved performance with deep soil improvement or piles has to be balanced against the additional cost.

Although engineers concentrate on the technical, the whole purpose of the built environment is to support society and people. Earthquake engineering is focused on reducing the impact of earthquakes on structures, but ultimately it is really about reducing the impact on people and society. One impact can be the mismatch between what people expect to happen and what really happens, between expectation and reality. Foundation related damage to one house inspected by the author was confined to a crack in the floor and some minor differential settlement; the building functionality was barely affected. Yet the owner considered that his house was "perfect" before the earthquake, and this expectation of "perfection" convinced him that the only outcome should be a full rebuild, financed, of course, with someone else's (insurance) money. As our current built environment presents at least a veneer of machine-like perfection, the expectation of society also increases. This has implications for disaster recovery, where these expectations can drive unnecessary repair and replacement. This in turn has wider implications, not just on the time and effort required for recovery, which has direct effects on personal and society well-being, but in the use of human and material resources, and ultimately, to climate change. The author has been unable to find any assessment of the carbon footprint of the rebuild, at nearly 0.5t/m^2 for concrete and steel buildings and 0.3 t/m^2 for residential housing, the Christchurch earthquake must have added a significant carbon footprint to the city's history.

It is my contention that as engineers, we have a role to educate our clients that some foundation deformation is likely to occur in a large earthquake, but that in many cases this will have limited impact on the building functionality. Instead, some of us have inflamed people's expectations or simply added to doubt and confusion. The following is excerpted from a Detailed Engineering Evaluation report on a commercial building which concluded that this structure was earthquake prone. Although not in the CBD (it is in Waltham), the building was on ground which performed similarly to the sites described above, with liquefaction predicted only at depth but an overall ground deformation.

> "We consider 150mm level variation from settlement to be a complete failure of foundation. Based on our measurements, the overall level variation is 128mm. Allowing for construction tolerances, the total settlement is approximately 108mm with a corresponding strength reduction of 108/150 = 72%. Therefore, an overall reduction factor of 0.28 will be applied to the buildings seismic capacity both along and across to account for earthquake damage".

Quite apart from the illogic of prorating bearing capacity with deformation, and the setting of 150mm settlement as "complete failure", the bulk of the foundation deformation was the result of a global settlement of the ground, and had nothing to do with the bearing capacity. The shallow strip foundations continue to support the building and transfer the building weight into the subgrade soils. At 37m long, the overall slope on the building was about 1 in 300, the buildings remained fully occupied and functional. Yet this statement caused considerable alarm to the building owner, triggered a series of additional reports, time delay and ultimately contributed to a relevelling of the building at considerable cost and disruption. Post-earthquake involvement with legal arguments over insurance and "as new" condition may have subtly affected engineers perceptions of acceptable building deformation, particularly where we have erred into becoming advocates for clients.

Ground deformation from earthquakes is a complex phenomenon. The tendency in post-earthquake Christchurch has been to carry out analysis for liquefaction, assess seismic loads on foundations and estimate settlement from bearing, and then pretend that we have predicted foundation performance in a future earthquake. The analysis is often confined to a simple two-dimensional geotechnical model confined to the site itself, as if liquefaction is the only driver of ground settlement. Rather, as the case studies have attempted to illustrate, deformation can result from the response of a complexly varying soil extending beyond the confines of a single site. We would do well to acknowledge the limits of our understanding, our models and numerical analysis, be a little more humble and accept that buildings may well end up out of level on even the best designed foundations. In some cases that may mean telling a client that the their performance specification is unachievable. If that understanding can be incorporated into designs that perhaps can be more readily relevelled if necessary, but more importantly into client and society expectations, then we will achieving a greater resilience. Resilience is not just how the structures survive an event, but how society responds, and that is as much mental and attitudinal as more tangible systems and management.

4 CONCLUSIONS

Deformations measured at three locations in the Christchurch CBD demonstrate that significant differential settlement can occur in buildings, not from any form of foundation failure, but simply from the foundation following the supporting ground as it deforms from a combination of seismic consolidation, the irregularity of seismic waves, variation in soil strengths in all three dimensions and liquefaction. These mechanisms can explain the deformations but tools to analyse them are approximate only. Emphasis on liquefaction analysis only can easily underpredict deformation and thus give unrealistic perspectives on future foundation performance. In some cases the differential movement experienced by foundations has been interpreted as failure, and even if not, has been deemed unacceptable when compared with societal expectations, even when the functionality of the building has been little if at all compromised. We would do better to look well beyond the individual site boundaries in assessing possible ground deformations, accept that current tools are foundation settlement are approximate only, and advise clients and society that some movement resulting in 1:200 – 1:300 floor slopes are quite possible.

Acknowledgements

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