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Geotechnical Research Objectives in Emergency Risk Management Plans

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ABSTRACT

Emergency risk management planning is obviously well developed in New Zealand and most of the developed economies in earthquake prone regions, giving the much-quoted "R-series": **R**eadiness, incorporating **R**escue, **R**ecovery and **R**esilience planning. This paper is proposing yet another 'R' in the readiness planning: **R**esearch. The inevitable earthquake aftershocks provide the opportunity to refine earthquake engineering knowledge, design methods and even assess the performance of earthquake related mitigation measures in a full scale, field setting under actual seismic loads.

This paper proposes that a geotechnical research programme is made ready for immediate post-earthquake implementation to support the evaluation of (amongst other aspects) the causes and effects of liquefaction and lateral spreading. Liquefaction and lateral spreading are identified as having clear uncertainties and omissions in our knowledge base. In particular, the proportion of co-seismic and post-seismic lateral displacement, the depths of sliding surfaces and the settlements of sites could be assessed with appropriate field instrumentation. The paper concludes with a discussion on a limited suite of field instrumentation installed in Christchurch following the 2010 Darfield Earthquake.

Introduction

On the morning of 22 February 2011, the authors of this paper were putting the finishing touches to a report entitled "Design criteria and performance requirements for lateral spread mitigation". The purpose of that report was to provide a design brief for engineers to design repair and strengthening measures at the numerous areas of lateral spreading in the Christchurch region that had resulted from the M_w=7.1 Darfield earthquake on 4 September 2010, some five months earlier. In fact, a trial repair had been designed for a typical lateral spreading site just north of Christchurch, and implementation of the construction work using stone columns had been "imminent" for the past two months. This work had not proceeded for a number of reasons, not least being debate and question amongst the geotechnical community regarding fundamental assumptions on which the report was based. In particular, the question of whether lateral spreading was co-seismic (dynamic) or post-seismic (static flow) indicated that designs could be either grossly conservative or inadequate. The questions remained but the design brief was an attempt to get past them with some pragmatic proposals.

By the afternoon of that day, the M_w= 6.3 "aftershock" or "Christchurch Earthquake" had resulted in more extensive liquefaction and lateral spreading than occurred in the September

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2010 earthquake, which stopped the design implementation. Aftershocks continued, notably on 4 months later on 13 June 2011 which in some areas resulted in re-liquefaction and yet more lateral spreading. In effect, aftershocks provide a full scale field trial which, if exploited, could assist in the resolution of uncertainties in liquefaction and lateral spreading fields.

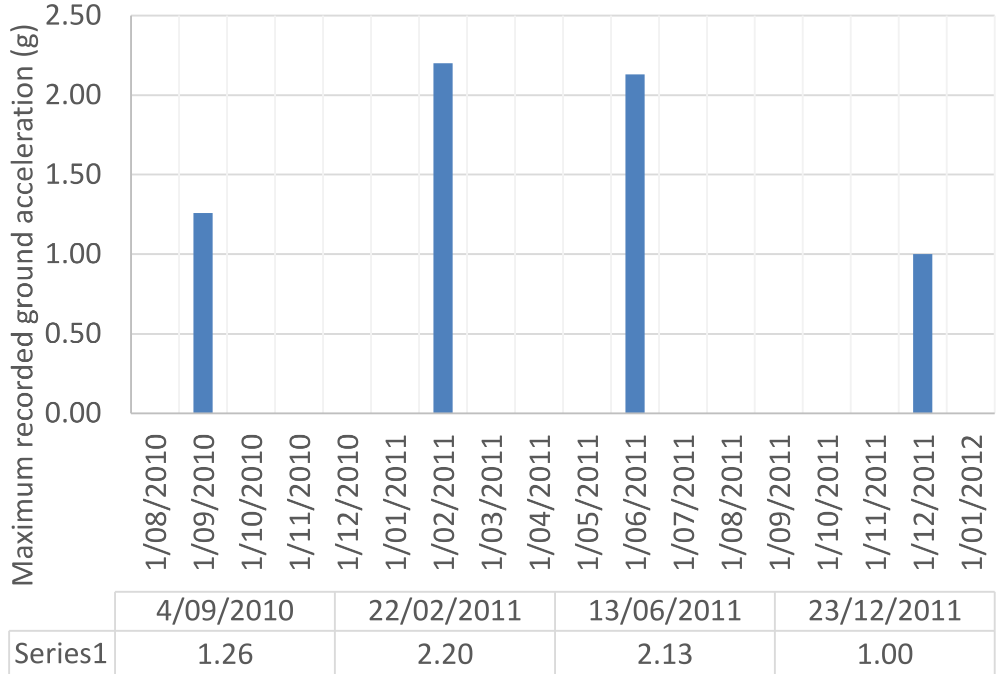

Figure 1: Timeline of major events in Christchurch earthquake series

The purpose of this paper is to highlight and identify the advantages of implementing monitoring, testing and possibly even construction trials immediately after major earthquakes. The intent of the work is to provide insight and possibly verification of design assumptions and analysis methods during aftershocks. Consequential aftershocks in Christchurch arrived five months and then four months after major earthquakes, which provided time to install instrumentation (see Figure 1). The eventual design of remedial works in the area could have been optimized and would have contributed to the state-of-the-art of Earthquake Engineering.

Most earthquake countries have emergency management plans. This paper proposes that value can be offered by including a funded research component that can be implemented immediately after major earthquakes. The balance of this paper considers some of the key discussions and questions being asked during the development of liquefaction and lateral spreading mitigation design measures, and then reviews the instrumentation installed in Christchurch after the September 2010 earthquake.

Lateral Spreading

The phenomena of liquefaction and lateral spreading are well known and subject of much attention (see Figure 2). However, to design protective measures against lateral spreading, it is necessary to address some fundamental questions, as yet still unresolved.

Figure 2: Example of settlement and lateral spreading along the Avon River, Christchurch

Driving Forces

Design requires evaluation of forces to be resisted and consideration of the following:

- What drives the lateral movement?
- Does lateral spread occur during shaking or after?
- Do inertial forces ("co-seismic") drive displacement?
- Are peak inertial forces coincident with full liquefaction?
- Are there static forces ("post-seismic") driving displacement?

There are numerous examples in the published literature that suggest that liquefaction, and lateral spread in particular, are primarily "post-seismic" phenomena (e.g. Marcuson et al., 1991). If this were the case, it would make design relatively simple. However, there are other statements in the literature (e.g. Dobry & Bazier, 1992; Arango, 2002) that suggest the "co-seismic" model, with liquefaction and lateral spreading initiated early in the shaking history. In this case, it implies that that inertial forces are applied to the land mass and thus it is necessary to assume the size of the moving land mass in order to design resisting forces.

Of particular relevance in this respect is the MCEER approach (MCEER/ATC, 2003) which provides a methodology for design of bridge abutments subjected to lateral spread conditions. The bridge abutments are assumed to be subject to inertial loads together with gravity loads that contribute to the driving forces. However, in Christchurch, the land is essentially flat, without any detectable slope to provide sufficient gravity driving force.

The Christchurch model, therefore is in effect a block of land, sliding on a level liquefied layer towards the "free face" (i.e. the river), driven either by inertial forces or by some static forces other than gravity. The consensus amongst New Zealand geotechnical community is that the former is the only safe (but perhaps conservative) way to proceed.

Size of Lateral Spreading Mass

Faced with this model, it is then necessary to have some way of determining the size of the ground mass being driven by the inertial forces. The bigger the ground mass, the greater the force required to resist it. Simple calculations show that the critical acceleration to cause movement of a block is independent of the length of the block (i.e. an "infinite" block model).

A certain amount of judgment is required in this respect but the following factors provide aids to making this assessment:

a Post Earthquake Crack Patterns: As part of the very detailed and extensive mapping exercise, ground crack patterns have been plotted for all damaged areas. Whilst the crack "density" and severity were obviously greatest near the river banks, the cracks were present in many places at considerable distance from the rivers, after several hundred metres. Clearly these distant cracks are more in the nature of "ground oscillation" consequences but, nevertheless, the patterns do not give an obvious answer. Is it safe to use the distances back to the first major crack?

b Strain Limits: It is reasonable to assume that a horizontal strain of 1:1000 in a typical building may be considered negligible and hence define an arbitrary limit of lateral spread. Information from crack patterns or Lidar may be available for this purpose. A typical "Bartlett & Youd" procedure (Youd et al. 2002) suggests that the lateral strain (slope of the displacement-distance plot) diminishes to 1:1000 at about 30 m to 50 m from the free face.

c Wavelength Effects: It is possible that the breaking up into blocks for lateral spreading is due to opposing motion at quarter to half wavelength distances. The predominant period for M_w=7.5 event is about 0.3 to 0.5 seconds. Shear wave or Rayleigh wave velocities are therefore likely to be about 150 to 200 m/s near surface. However, coherence effects are complex and tend to be governed by wave velocities at depth, and hence the maximum distance to tension zones from the free face is likely to be about 125 to 250 m.

An obvious question to ask is why do displacements decrease with distance from the face? This would imply a "shuffle-back" effect whereby the first block moves early in the shaking history, thus providing opportunity for the next to move, but with a delay. Mitigation designs, however, typically consider a single, coherent block slide failure.

Depth of Sliding Surface

Lateral spread is often characterised as a block of non-liquefiable land or "crust" sliding on a liquefied layer (e.g. Dobry & Bazier, 1992, Boulanger et al., 2007). Publications appear to indicate that there is a deformation profile through the liquefied layer (e.g. Dobry & Bazier, 1992; Tokimatsu & Asaka, 1998; Yasuda & Berrill, 2000). However, in order to compute inertial forces, it is necessary to define the sliding mass, and hence the rigid block model is convenient. For this analysis, the depth to sliding surface is generally assumed to be controlled by both the depth to liquefied layer and the height/depth of the free face. Observations at

Christchurch suggest some small toe "heave" in the river beds, so that a sliding surface taken just below or at deepest channel level seems reasonable.

Liquefied Residual Strength

It is recognised that even fully liquefied soils have some residual strength. Recent discussion and recommendations for assessing this are given by Idriss & Boulanger (2008). In our context at Christchurch, the CPT-based method is probably most appropriate, the chart for which is given by Idriss & Boulanger (2008). For the shallow depths to liquefaction in Christchurch, the residual strength would normally be in the range of 0.08 to 0.1 times the vertical effective stress, or around 4 to 6 kPa.

However, the primary question here is what proportion of the strength loss in the liquefied soil coincides with the peak inertial forces? Any early peak acceleration may have only partial strength loss, and a late peak would not be fully transferred to the sliding land mass. Some materials (for instance the low plasticity Adazapari silts discussed in Bray and Sancio, 2006) dilate on shear. This type of behavior may have substantial implications for assessing the magnitude of lateral spreading deformations.

Post-seismic Lateral Spread

Whilst the literature often suggests "co-seismic" lateral spread assumption for design, there have been a number of published observations that lateral spread is predominantly "post-seismic" (e.g. Marcuson et al., 1992). Post-seismic lateral spread was also observed in Christchurch by a colleague of the author on June 13, 2011 when two after-shocks (M_w = 5.5 and 6.3) caused yet more liquefaction. The observed lateral spread occurred suddenly and dramatically, but some time after the shaking for the second and longer event. The spreading was preceded by head-high "geysers" and sand ejecta. This suggests the "impedance" model whereby the crust causes high water pressures which then break through and impose high lateral pressures on the sliding block.

If post-seismic deformation could be confirmed at certain sites as the primary mechanism of failure, then inertial loads could be ignored for mitigation design. This would significantly simplify and reduce the scale of lateral spread mitigation measures.

Passive Pressure Limit on Lateral Force

Dobry & Bazier (1992) proposed a three-block soil model for analysis of lateral spread, the leading block being the passive wedge. If this model were applied to our design situation, with inertial loads driving the spreading ground and resisted by a treated block, it would imply that the maximum possible force would be the passive resistance. This would be a convenient model as it is independent of the mass of ground influenced by inertial forces and only depends on the strength and depth of the crust. The disadvantage of this model is that it implies that deeper crusts require more extensive resisting measures, which appears to be counter-intuitive.

What would have helped?

The following instrumentation could have provided some insight into the uncertainties and questions discussed above:

- Depth of sliding surface: measurement of lateral displacement profiles with depth, either after the sliding is completed (inclinometers) or in real-time (shape arrays).
- Pore-pressure generation: Pore pressure transducers (divers) giving close to real time pore pressure measurement and therefore indicating strength loss in relation to ground shaking history
- Survey monitoring pins or monuments to track settlements and lateral deformations
- Digital high definition video of the site to show any liquefaction or lateral spread that may occur
- Site based seismographs to provide actual acceleration records from aftershocks

There may be other practical measures that could be worked out for a formal plan, depending on the cost and benefit of each instrument.

Other Geotechnical Earthquake Issues

Whilst liquefaction and lateral spreading are the current priorities, other geotechnical issues may dominate for other locations at the next significant earthquake. Obviously, there are many aspects of earthquake engineering that would benefit from pre-planned research objectives, and some of these would merge with the realm of geotechnical engineering. The plan for instrumentation of buildings by GNS in already well developed (Uma et al., 2011) and such work may help with evaluation of foundation performance. This would need to be considered further at some time during development of the plan. Some other geotechnical issues that carry significant uncertainty or lack consensus amongst geotechnical community may include:

- Soil pressures on embedded structures such as basements and tunnels: These structures move with the soil so that earthquake pressure increase may be negligible or possibly governed by kinematic effects.
- Site effects generally but non-linear effects in particular with soft soils: the US-based codes permit reduced design actions at high levels of shaking (with their factors F_{PGA}, F_a, F_v) but NZS1170.5 does not. This 1170.5 restriction may be unrealistically conservative at high design levels.
- Landslides, slope stability, and in particular topographic amplification: NZS1170.5 gives no specific requirement nor guidance on how to allow for topographic amplification. Eurocode 8 gives such guidance but some other publications suggest even higher provisions (e.g. Ashford & Sitar, 2002).

The above examples are more in the nature of pure research. Clearly, instrumentation and monitoring of known existing landslides would also be of value in design of stabilizing measures.

Examples from Christchurch

Instrumentation was installed after the major earthquake in Christchurch and is summarized in Table 1 below. The monitoring data is still being reviewed and analyzed, with an intention to publish the results and conclusions in the future. The authors experience is that a more planned and structured instrumentation regime would have been beneficial in targeting sites and instruments to provide results that most closely address specific questions to be answered.

Table 1. Summary of Instrumentation.

Instrumentation	Commentary
4 Inclinometers	The inclinometers were installed in areas of lateral spreading and tracked lateral spreads of up to 40 mm.
2 Shape arrays	These provide realtime deformation data in two locations of severe lateral spread.
103 survey monuments	Three separate networks were set up in areas affected by liquefaction after September 2010 to track future movement and settlements.
5 Solinst Levelogger piezometers	These were located in separate location and were set up to measure pore pressures at 15 second intervals. This allows the piezometric response of during earthquakes to be monitored.

The survey network and the piezometers proved the most useful, the former for determining the patterns of lateral spreading and the latter for providing insight into generation and dissipation of pore pressures in real time. Figure 3 shows an example for one site (Bexley) during the June 2011 pair of aftershocks that resulted in liquefaction and lateral spreading.

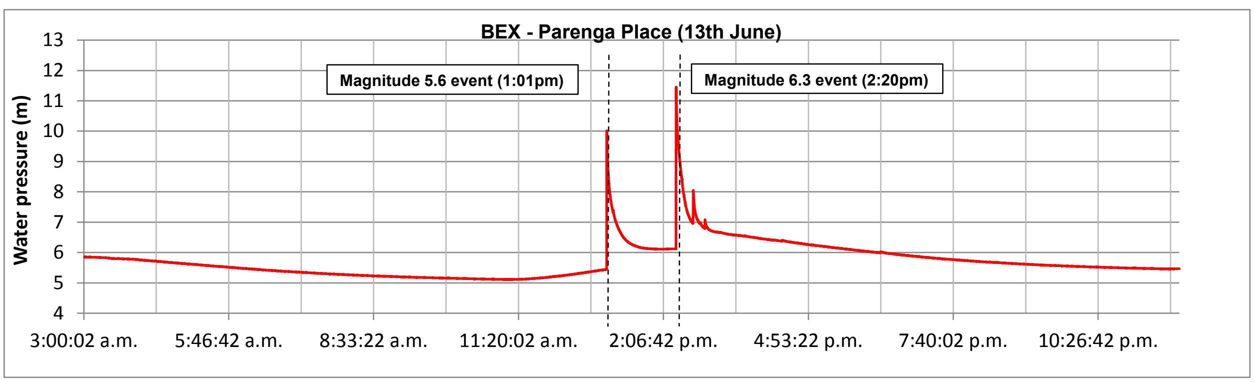

Figure 3: Piezometer readings at 5m depth for 13 June 2011 at Bexley

Trial Mitigation Measures

As discussed above, there are many uncertainties relating to cause of lateral spreading and the factors that would influence design. Depending on the judgment calls that are required with the present state of knowledge, designs to prevent lateral spreading could vary considerably in complexity and cost.

Usually, mitigation against lateral spreading involves ground treatment of some sort within a strip of land adjacent to the free face. Ground treatment generally involves stone columns or columns of stronger and more rigid material such as provided by deep soil mixing (DSM) or piles. There is debate relating to the required column density and width.

Following a major earthquake, trial designs could be prepared for locations that had suffered some lateral spreading in that event, contractors selected and construction work carried out in as short a period as possible. The proposed plan should provide for a means of procuring and implementing concepts, including identifying specialist designers, contractors and sources of plant and materials.

Discussion and Conclusions

The purpose of this paper is to propose that a Research objective is included in earthquake emergency management plans. This would improve both recovery and resilience. The proposal is based on the following axioms:

- Major earthquakes have aftershocks, which can be used to research and test physical principles, design criteria and trial mitigation measures.
- A detailed plan is required so that it can be implemented immediately after a major initial event, despite the competing priorities and imperatives associated with Rescue and Recovery.
- The detailed plan should include a list of objectives, identification of expected instrumentation with procurement sources and cost estimates, lists of potential designers and constructors for both installation of research instrumentation and trial mitigation measures, and a management structure for implementation and delivery.

The plan should be prepared by a committee comprising academics, engineering practitioners, constructors and emergency management professionals.

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