Design of a contiguous bored pile wall to reduce lateral spreading potential; force based and displacement based approaches

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ABSTRACT

Liquefaction induced lateral spreading associated with the Canterbury earthquakes of 2010 and 2011 has resulted in significant damage to numerous riverside properties in Christchurch.

A number of methods for mitigating lateral spreading risk exist; these often involve ground improvement adjacent to the free face.

Preliminary design using a contiguous bored pile wall to retain liquefied soil and reduce the magnitude of seismically induced lateral spreading has been carried out for a site in Christchurch. Both force based and displacement based design methods have been used for this design. A summary of each method is presented, followed by discussion of the relative merits of each approach and parameter sensitivity.

1 INTRODUCTION

This paper summarises two different approaches used in design of a bored pile wall to mitigate liquefaction induced lateral spreading. Design has been carried out using force based and displacement based approaches, for a site adjacent to the Wairarapa Stream in Christchurch.

Site conditions and past site performance are summarised, followed by a description of the design philosophy and methodologies, and a discussion on the relative benefits and limitations of the approaches used.

2 BACKGROUND

Liquefaction and lateral spreading occurred at the subject site as a result of the 04 September 2010 and 22 February 2011 earthquakes. The total cumulative lateral displacement was assessed to be approximately 300mm at the riverbank, based on the cumulative width of cracks measured on site. Much of this displacement was expressed in a large crack running through the centre of a residential dwelling, located approximately 15m from the riverbank. A number of smaller cracks were observed at distances up to 40m from the riverbank.

Geotechnical investigations carried out at the site indicate that the natural ground conditions comprise:

- 3.0m to 4.0m of alluvial sandy gravel (SPT 'N' typically 15 to 25), overlying,
- 5.0m to 6.0m of alluvial silt, sandy silt, and silty sand (SPT 'N' typically 5 to 15), overlying,
- Unconfirmed thickness (>9.0m) of dense alluvial sandy gravel (SPT 'N' typically greater than 30).

The inferred generalised subsurface profile for the subject site is summarised in Figure 1.

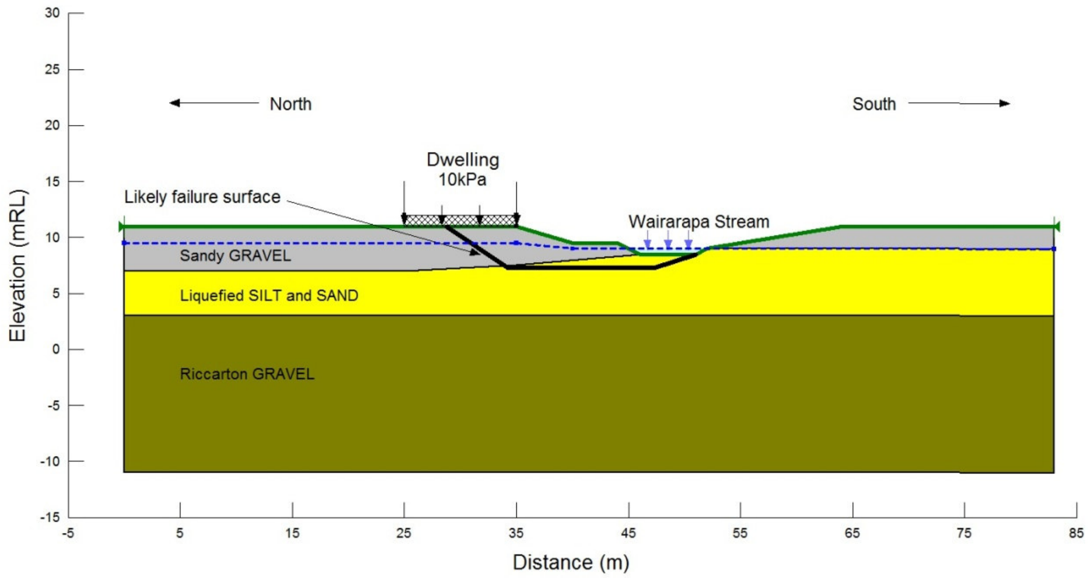

Figure 1: Subsurface profile

3 BACK ANALYSIS

A back analysis of the lateral spreading which occurred at the site in the 04 September 2010 and 22 February 2011 earthquakes was undertaken. The purpose of this was to confirm that the assumed profile of the sliding surface was realistic, and to calibrate the shear strength of the sliding surface (i.e. shear strength of liquefied material).

The back analysis assumed the following parameters to represent the 04 September 2010 and 22 February 2011 earthquakes:

- 04 September 2010: Moment Magnitude 7.1, peak horizontal ground acceleration 0.22g;
- 22 February 2011: Moment Magnitude 6.2, peak horizontal ground acceleration 0.31g.

Using the method of Idriss & Boulanger (2008), approximately 70% of the alluvial silt and sand material encountered between 3.0m and 9.0m depth was assessed to be liquefiable during the 04 September 2010 earthquake and 22 February 2011 earthquakes. The residual shear strength of this liquefied soil (τ/σ ratio) was indicated to be in the range of 0.05 to 0.15 based on empirical relationships with CPT cone resistance (Idriss & Boulanger, 2008).

The inferred mechanism of failure may be described as a block of the non-liquefiable gravel crust sliding on liquefied material, with a slip surface located at approximately 3.0m to 4.0m depth (i.e. at or just below the level of the base of the stream channel) (Refer Figure 1).

The analysis methods used in the back analysis are summarised in the following sections.

3.1 Empirical analysis

A lateral spreading back analysis using the empirical method published by Youd et al., (2002) was undertaken. The results of this analysis indicated displacements at the riverbank as follows:

- 04 September 2010: 200 – 400mm, and,
- 22 February 2011: 100 – 200mm.

These displacements are generally consistent with displacements inferred from measuring crack widths on site.

3.2 Flow failure

The software package SLOPE/W (Version 7.2, GeoStudio, 2007) was used to model the site (refer Figure 1). The shear strength of the liquefiable material was set as $\tau/\sigma=0.05$ (i.e. lowest probable strength based on empirical relationships with CPT cone resistance) in order to assess whether a flow failure (i.e. displacement occurring without horizontal acceleration) is possible.

A factor of safety of slightly less than 1 for distances of up to 15m from the riverbank, and greater than 1 for distances greater than 15m from the riverbank was obtained using this method. This indicates that some of the displacement at the site may have occurred after earthquake shaking had stopped.

3.3 Newmark sliding block

For the purposes of the back-analysis, and subsequent design, the non-liquefiable gravel crust was considered as a rigid sliding block (Newmark, 1965). The empirical method published by Jibson (2007) was used to assess yield acceleration values corresponding to peak horizontal ground acceleration and ground displacements measured at the site for each earthquake.

3.3.1 Width of sliding block

Slope modelling was undertaken for a range of different sliding block widths. Widths of 15m and 40m (measured from the riverbank) were considered, in order to represent the location of the main crack (15m) and the crack observed farthest from the riverbank (40m). A sliding block width of 100m was also considered. This 100m block was assessed to be the maximum possible size of soil block which is expected to move in phase during earthquake shaking (i.e. ¼ of approximate p-wave length). At distances greater than 100m from the riverbank, the full width of the sliding block is not likely to be in phase. The back analysis of shear strength (refer Section 3.3.1) assumed a block width of 40m. The forward analysis (refer Section 4) assumed a 40m block width with a sensitivity check of 100m.

3.3.2 Sensitivity to time history assumed

The method published by Jibson (2007) calculates a mean displacement, along with lower bound and upper bound displacements (corresponding to one standard deviation above or below the mean). This range represents the variability in the shape of earthquake time-history records considered in the empirical model used by Jibson. As the shape of the design earthquake time-history is not known, a conservative approach has been adopted; mean displacements have been used when calculating yield acceleration values as part of back analysis, and upper bound displacements are used for forward analysis.

3.3.3 Failure surface shear strength

The shear strength of the sliding surface (τ/σ) based on the Newmark sliding block approach was back-calculated as 0.14 to 0.17. This shear strength is at the upper end of the range of residual shear strength of liquefied soil assessed from empirical relationships with CPT cone resistance (0.05 to 0.15). This could reflect the development of excess pore water pressure and corresponding reduction in shear strength of liquefiable material throughout the duration of the earthquake. The shear strength calculated from the back analysis could be considered as an average over the duration of earthquake shaking, rather than residual liquefied shear strength.

3.3.4 Discussion of Newmark Sliding Block

Important limitations of the Newmark sliding block model include the following. These were considered during our analysis.

1) The liquefied residual shear strength calculated from the back analysis may be unconservatively high for use in forward analysis if the earthquake used for the back analysis has lower magnitude compared to the design earthquake. This could be accounted for by adopting a lower bound shear strength value. Alternatively, the liquefied residual shear strength estimated using empirical relationships with CPT cone resistance could be assumed for all analyses, however this approach may be over-conservative.

2) The liquefied residual shear strength at the end of earthquake shaking may be significantly lower than the average shear strength assumed throughout earthquake shaking. When using the Newmark sliding block approach in design, it is necessary to check the post earthquake static stability of the system assuming the lowest probable liquefied residual shear strength.

3) The Newmark sliding block model is based on the assumption that the majority of lateral spreading displacement occurs during earthquake shaking. Anecdotal evidence and observations made by geotechnical engineers throughout the Canterbury earthquake sequence of 2010 and 2011 indicate that a significant portion of lateral spreading movement occurs after earthquake shaking has stopped. This is consistent with the possibility of flow failure (refer Section 3.2), and the need to check post-earthquake static stability.

The Newmark sliding block model includes a number of other approximations and simplifications which may not represent actual conditions. These include:

- Soil assumed to move as a single rigid block rather than a series of non rigid blocks. Actual failures are observed to occur in a series of scallops.
- Movement is assumed to occur on a distinct failure surface. Actual movement may occur by straining through a significant depth of liquefied soil.
- Shearing is assumed to be rigid plastic. Actual conditions may be non linear elastic plastic.
- Acceleration response of the surface (PGA) is considered rather than an in ground response.

Despite the limitations discussed above the back analysis indicates that the Newmark sliding block analysis can model observed outcomes and thus has value as a design tool (with care). Bazair et.al. (1992) undertook a similar back analysis of lateral spread displacements which occurred during the 1987 Superstition Hills earthquake (California). This back analysis also supported Newmark sliding block as a valid model. For more detailed assessment dynamic analyses should be considered. It is noted that these analyses will also include a number of assumptions and simplifications.

4 FORWARD ANALYSIS

A bored pile wall located between the dwelling and the river channel was investigated as an option to mitigate future lateral spreading. Ground improvement adjacent to the river channel was assessed to be infeasible due to a lack of a suitable width of ground, and the presence of protected trees over much of the site. The following sub-sections describe the preliminary design of the pile wall.

4.1 Design objectives

To meet the New Zealand Building Code requirements buildings are required to be designed to:
- Avoid collapse during a large earthquake (Ultimate Limit State, 500 year return period); and,
- Not suffer significant damage and retain amenity following a moderate earthquake (Serviceability Limit State, 25 year return period).

In response to these Building Code requirements a design objective of limiting ground displacement to less than 200mm in an Ultimate Limit State (ULS) earthquake (Moment Magnitude of 7.5, peak horizontal ground acceleration of 0.35g) was nominated.

Without the pile wall, lateral spreading analyses using the Newmark sliding block model and the empirical method published by Youd (2002) indicated 0.5 to 2m displacement may occur at the riverbank in a future ULS design earthquake (M_w 7.5, PGA 0.35g). This estimated displacement is significantly higher than that observed and assumed for the back analysis. This highlights the sensitivity of the empirical analysis methods to the magnitude of earthquake.

Force based and displacement based design approaches were used, in order to assess the relative merits of each approach and develop a methodology to be used in future design projects.

4.2 Force based approach

1. The magnitude of the force imposed on the wall by laterally spreading soil was estimated by three approaches:
   a) Considering the soil behind the wall as a hydrostatic fluid with a weight of $18kN/m^3$ (Gurley & Nicholls, 1982), with the force on the wall resulting from a hydrostatic stress distribution (coefficient of lateral pressure K=1). No acceleration was applied to this model. This approach is not considered to be correct for this site as the majority of soil force on the wall is likely to result from the non-liquefied gravel crust. However, this approach may be appropriate for other sites where the majority of force imposed on a wall is likely to result from fully liquefied soil. As such, it is included for comparison purposes only.
   b) Modelling the wall as a pile element in the SLOPE/W model, applying a seismic coefficient of 0.35g, and varying the shear capacity of the pile to achieve a factor of safety of 1.0. Non-liquefied soil conditions were assumed. Soil strengths of c=0 and $\phi=35°$ in the gravel crust and c=0 and $\phi=28°$ in the underlying silt and sand were assumed. The purpose of this step is to account for the possibility that a Coulomb wedge or other type of failure may form early on during an earthquake, before liquefaction and the associated lateral spreading type failure has fully developed.
   c) As above, assuming liquefied soil conditions. Soil strengths of c=0 and $\phi=35°$ in the gravel crust and $\tau/\sigma=0.15$ in the underlying liquefied soil were assumed. A seismic coefficient of 0.35g was applied, and a sliding block width of 40m assumed for this analysis.
   d) The maximum passive pressure that the sliding block can apply to the wall was also considered. (Note in this case in order to apply this force the block width would have to be greater than 100m, hence this was not considered further).

2. The software package WALLAP (Geosolve, Copyright 2012, D.L. Borin) was used to model the bored pile wall. The force imposed by laterally spreading soil (from Step 1) was applied as a load, and the wall pile diameter varied to limit wall deflection to 200mm or less.

4.3 Displacement based approach

1. The empirical method published by Jibson (2007) was used to assess yield acceleration values corresponding to 200mm of displacement for the ULS design earthquake.
2. The SLOPE/W model was used to model the wall as a pile element. The shear capacity of the pile element was set to achieve a factor of safety of 1.0, subject to the yield acceleration calculated in Step 1. A sliding block width of 40m and a shear strength of the liquefied soil of $\tau/\sigma=0.15$ were assumed.
3. The software package WALLAP was then used to determine the required pile size to provide the resistance calculated in Step 2, with deflection equal to 100mm (i.e. wall restraint develops with displacement (Palmer & Jacka, 2008) up to 200mm, average restraint taken as that which is provided at 100mm displacement).

4.4 Design outcomes

The size of the bored pile wall required to meet the design objective varied significantly depending on the design approach used. The design outcomes are summarised in Table 1.

Table 1: Design outcomes

Design approach	Required pile diameter
Force based (a)	750mm at 750mm centre to centre spacing
Force based (b)	600mm at 600mm centre to centre spacing
Force based (c)	1600mm at 1600mm centre to centre spacing
Displacement based	750mm at 750mm centre to centre spacing

The post-earthquake static case (refer Section 3.3.1) was checked for all cases but was not found to be critical to design.

5 PARAMETER SENSITIVITY

The parameters used in the displacement based design were as follows:

- Shear strength of liquefied soil (τ/σ): 0.15
- Sliding block width: 40m
- Wall capacity at 100mm displacement: 110kN/m wall (750mm dia. wall)
- Jibson (2007) displacement output: upper bound (i.e. mean plus one standard deviation)

A sensitivity analysis was carried out on the displacement based approach by varying the assumed parameters and calculating the resultant change in wall displacement. The sensitivity analysis is summarised in Table 2.

Table 2: Sensitivity analysis

Scenario	Calculated total displacement (mm)
Design	150
Reduced liquefied shear strength reduced to 0.10	275
Sliding block width increased to 100m	240
Wall capacity at 100mm displacement reduced to 55kN/m wall	240
Jibson (2007) mean displacement	55

It is noted that increasing the sliding block size results in lower yield acceleration and a corresponding increase in calculated displacement. This differs from what would be intuitively expected. Hubert & Ignatius (2000) noted that this is partly due to more variation of inertia within the soil mass for a larger sliding block. In other words, even for a mass of soil which is moving in phase, the rate of movement varies within the soil mass. Hubert & Ignatius used two-dimensional site response analysis to compute time histories for a range of block sizes, and used Newmark's approach to show that displacement decreases with increasing block size. This indicates that the 100m wide sliding block may not be a realistic scenario.

The increase in calculated total displacement resulting from reducing wall capacity and liquefied shear strength is significant. The analysis is sensitive to changes in liquefied shear strength and wall capacity, and as such, conservative values should be adopted for these parameters.

6 CONCLUSIONS AND RECOMMENDATIONS

The relative benefits and drawbacks of the two design methodologies used are summarised in Table 3.

Table 3: Force based and displacement based methodology comparison

Design methodology	Advantages	Disadvantages
Force based	• May be appropriate for Serviceability Limit State design or applications where limiting/preventing displacement is critical	• Likely to be over-conservative for ULS design • Relies on accurate estimation of force imposed on wall by laterally spreading soil – difficult to estimate accurately • All displacement is assumed to occur during earthquake shaking
Displacement based	• Reflects that preventing all displacement may not be realistic or economic • Designer can control level of conservatism by selecting appropriate geotechnical parameters and designing to upper/lower bound or percentile displacements	• Includes the limitations and assumptions of a Newmark sliding block analysis. These may not accurately reflect the actual process of lateral spreading (refer Section 3.3.4).

The following conclusions and recommendations are made based on lessons learnt undertaking this work:

- Newmark sliding block includes many assumptions which may not reflect the actual process of lateral spreading. However, the block analysis indicated that it can model observed outcomes and this has value as a design tool (with care).
- Back analysis based on performance of the site in past earthquakes was valuable to confirm assumptions such as the location of the sliding surface, and to refine geotechnical parameters such as the post-liquefaction shear strength.
- Liquefied residual shear strength calculated from back analysis may be higher than that calculated using empirical relationships with CPT cone resistance. Careful consideration and engineering judgement should be applied when assessing the appropriate post-liquefaction shear strength for use in design.
- The force based approach may be appropriate for Serviceability Limit State applications where there is a requirement to prevent any displacement from occurring.
- A displacement based approach may be appropriate for analysis and design of this type of system, particularly for Ultimate Limit State design applications where some displacement is acceptable.

- Design approaches need to account for the fact that wall restraint develops with displacement.
- The post-earthquake static stability of the system should be checked, using the lowest probable post-liquefaction shear strength.
- Multiple methods should be considered for assessment of lateral spreading type problems. Ultimately, engineering judgement must be applied in order to assess the most likely failure modes and appropriate design solutions.

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