Assessment of lateral spread potential used smeared strengths of liquefied/ non liquefied soils.

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ABSTRACT

Liquefaction-induced lateral ground displacements (lateral spread) are typically assessed using empirical methods where a continuous weak (liquefied) layer is expected. These methods may not be appropriate for variable ground conditions where continuous weak (liquefied) layers are not expected. End-tipped reclamation fill materials on the Wellington Waterfront are highly variable, and the materials tend to vary by zones rather than continuous sub-horizontal layers. Numerical methods in conjunction with a ‘smeared’ soil strength offered a possible means of evaluating lateral spread potential in this situation. This paper presents the process adopted to estimate lateral spread potential using smeared strengths of liquefied soils for a site on the Wellington Waterfront. The determination of smeared strengths was based on a weighted average strength of soils divided up by soil behaviour type (Ic) from CPT testing and depth. The smeared strength approach was considered appropriate for predicting lateral spread displacements at locations relatively distant from the reclamation edge (i.e. A distance of > 2 times the depth to the base of the liquefiable materials). At locations within proximity of the reclamation edge there is a risk of a local zone of weaker fill materials dominating the overall strength, so the smeared strength may not be representative. Sensitivity analyses and engineering judgement were applied to the results of the lateral spread predictions when considering the potential range of ground displacements for the site.

1 INTRODUCTION

Lateral spread as a consequence of strong earthquake shaking is known to be a risk along the Wellington Waterfront within the reclamation fills. Reclamation fills along the Wellington Waterfront vary by age, material type and construction method. This paper considers reclamation fills constructed by process of end-tipping, resulting in a highly variable reclamation fill where the materials tend to vary by zones, rather than continuous sub-horizontal layers.

Empirical methods, Youd et al. (2002) and Zhang et al. (2004), are routinely used to predict liquefaction induced lateral ground displacements (lateral spread). These methods consider that lateral spread will occur along a continuous weak (liquefied) layer. These methods are not appropriate for use within an end-tipped reclamation fill where a continuous weak layer is not expected. However, the risk of lateral spread at such sites cannot be discounted.

This paper presents the process of determining a ‘smeared’ soil strength considering zones liquefied and non-liquefied soils within the reclamation fill. This process allows the use of a Newmark Sliding Block assessment to predict lateral ground displacements. There is a risk that a locally weak zone of material may occur close to the reclamation edge, and this should be considered in the assessment.

2 GROUND MODEL

2.1 Soil Profile

Table 1 presents a generic ground model for discussion within this paper. This is based on a reclamation fill constructed in the 1970’s from residual soils excavated during local construction activities.

Table 1: General soil profile

Layer	Geological Unit	Soil Description	Layer Thickness (m)	Typical SPT N (blows/ 300 mm)	Typical CPT qc (MPa)
1	Reclamation Fill (1970s)	Variable end tipped fill. Varying from soft sandy SILT to loose silty sandy GRAVEL.	12	4 to 5	2 to 6
2	Marine Deposits	Very soft to soft sandy and clayey SILT Interbedded with loose silty SAND and shell fragments.	0.5	3 to 7	1
3	Alluvium	Medium dense to very dense silty SAND and GRAVEL With occasional lenses of stiff SILT/CLAY.	5	30 to 50+ 7 to 15	5 to 30
4	Basement Rock	Greywacke	N/A	50+	N/A

The design groundwater level is assessed to be 2 m below ground level.

2.2 Liquefaction effects

The reclamation fill and marine deposits have been assessed to typically vary between:

- Clay-like materials that are not susceptible to liquefaction;
- Low plasticity silt, sand and gravel that is assessed to be susceptible to liquefaction.

As a result of strong earthquake shaking, widespread liquefaction of the susceptible soils could occur. Lateral spread, sand boils and settlement were identified as possible consequences of liquefaction at the site. Cyclic softening of soils assessed as non-susceptible could be expected.

Liquefaction effects within the alluvium deposits are assumed negligible and not discussed within this paper.

3 DETERMINATION OF 'SMEARED' STRENGTH

3.1 Suitability of 'smeared strength'

The application of a smeared strength is considered appropriate for the following reasons:

- The reclamation fill was placed by the end-tipping of material. This end-tripping and variability of material supply could be expected to produce fill varying in zones rather than continuous sub-horizontal layers.
- The CPT results at the site indicate variability of reclamation fill material (sand like to clay like) by location rather than in continuous sub-horizontal layers.
- Zones of susceptible soil could liquefy. The associated pore water pressure is not expected to dissipate into and adversely affect the strength of adjoining non-susceptible soils because of the relatively low permeability of non-susceptible cohesive soils.
- Liquefaction assessment indicated that the cumulative thickness of liquefiable fill to represent between 25% to 50% of the total thickness of the fill below the groundwater level. There was no indication of parts of the site having greater thickness of liquefiable soil.
- CPT results do not show a clear demarcation between the reclamation fill and marine deposits. The marine deposits have been assessed to vary between sand like and clay like and have been included within the smeared strength calculated for the reclamation fill.

3.2 Division of soils

A smeared strength for the reclamation fill can be assessed for various depth increments, to allow for variations in overburden pressure. The depth increments should be selected by the Geotechnical Engineer considering the total depth of the reclamation fill and liquefiable soils at the site.

For each depth increment, the reclamation fill is divided up by soil behaviour type (Ic) from CPT testing, The three Ic categories proposed are:

- Ic < 2.6: Sand-like materials
- 2.6 < Ic < 2.95: Marginally cohesive materials
- Ic > 2.95: Clay-like materials

3.2.1 Soil behaviour type: Ic < 2.6

For soils with Ic less than 2.6 (gravel to sandy silt), a residual liquefied soil strength was assumed. The residual liquefied shear strength (Sr) of the soils was calculated using Equation (1).

$$S_r = \left(\frac{S_r}{\sigma_v'}\right)\sigma_v' \qquad (1)$$

where S_r = residual undrained shear strength of liquefied soil; S_r/σ_v' = based on the relationship with q_{1N-cs} ; and q_{1N-cs} = normalised clean-sand equivalent cone tip resistance.

The relationship between S_r/σ_v' and q_{1N-cs} is based on average of the published methods by Idriss and Boulanger (2007), Kramer and Wang (2015) and Weber (2015).

3.2.2 Soil behaviour type: 2.6 < Ic < 2.95

Soils with Ic between 2.6 and 2.95 are assumed not to liquefy and therefore a strength is assessed for these soils without liquefaction effects. These soils are assessed as having either frictional or cohesive behaviour. Their shear strength assuming frictional behaviour was calculated using Equation (2) and assuming cohesive behaviour using Equations (3) and (4). The frictional assumption resulted in a lower shear strength and therefore the frictional strength was conservatively assumed. For the purposes of this assessment, it has been assumed that the undrained shear strength (Su) is equivalent to the effective shear strength of the soil (τ').

$$\tau' = \sigma_v' \tan(\varphi') \tag{2}$$

where φ' = friction angle of these materials.

$$S_u = q_c / N_k \tag{3}$$

where S_u = undrained shear strength of non-liquefied soil; q_c = cone tip resistance; N_k = cone coefficient taken as 15.

Cohesive soils were assessed to be susceptible to cyclic softening, therefore a reduced undrained shear strength for clay was determined using Equation (4).

$$S_{ur} = 0.8 S_u \tag{4}$$

3.2.3 Soil behaviour type: Ic > 2.95

Soils with Ic greater than 2.95 are assumed to be cohesive and not liquefiable in nature. Therefore a strength is assessed for these soils without liquefaction effects. The undrained shear strength are assessed as a cohesive material using Equations (3) and (4).

3.3 Design smeared strength

The cumulative thickness (%CT) of each soil type (Ic range) was assessed as a percentage of the total thickness of each depth interval.

The design smeared strength for each depth increment for each CPT was assessed using a weighted average of each soil behaviour type strength presented in Equation (5).

$$S_{smeared} = (\%CT_{Ic<2.6})S_r + (\%CT_{2.6<Ic<2.95})\tau' + (\%CT_{Ic>2.95})S_{ur} \tag{5}$$

Smeared strength results from a number of investigation points should be assessed and averaged to determine a design smeared strength for use in lateral spread assessment. Individual smeared strength values which are judged to be 'outliers' in the results should be excluded from the average for the design smeared strength.

It is noted at shallow depths (low overburden pressure), that the clay-like soils (Ic > 2.95) dominate the smeared soil strength as this is not dependent on overburden pressure. It is for that reason, that it is recommended to use a smeared strength with units of kPa in lateral spread assessment rather than a smeared strength normalised by overburden.

4 METHOD OF LATERAL SPREAD ASSESSMENT

The smeared strength approach cannot be applied to empirical methods of predicting lateral spread (e.g. Youd et al. (2002) and Zhang et al. (2004). But it could be applied in dynamic finite element modelling methods or Newmark Sliding Block (NSB) methods in conjunction with pseudo static limited equilibrium stability analyses. Here we discuss application of the smeared strength to a NSB approach.

4.1 Newmark Sliding Block (NSB) approach

The key inputs to this assessment are presented in Figure 1 and include:

- Slope Geometry
    - Free face height, H

- Ground surface slope S

- Soil Properties
  - Non-liquefied soil strength for materials above groundwater level.
  - Smeared strength of liquefied and non-liquefied soils as presented in Section 3.3 above.
- Earthquake magnitude, M and peak ground acceleration (PGA).

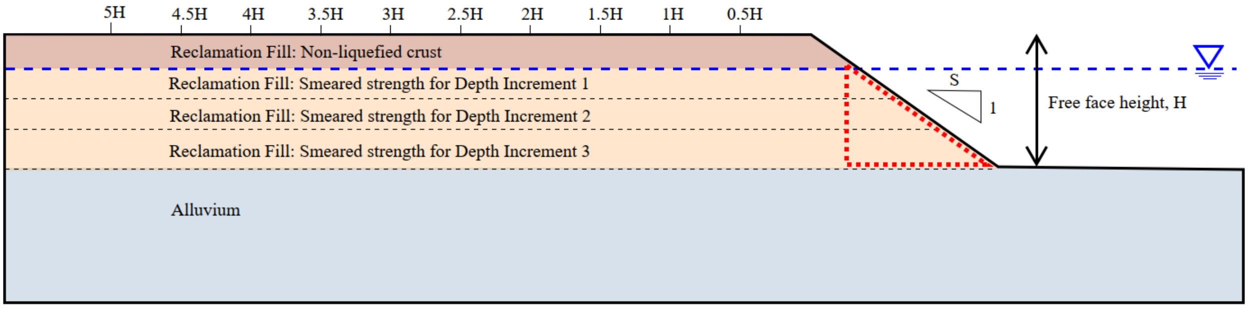

Figure 1: Lower bound 'smeared' strength soil profile and geometry,

Within the zone beneath the batter slope (indicated by red dashed line in Figure 1), the smeared strength should be appropriately reduced to account for the reduced overburden pressure.

4.2 Sensitivity

Because of the simplifications and uncertainties of lateral spread prediction, and particularly NSB, multiple NSB methods should be considered to assess the seismic displacement from the yield acceleration. Here we have considered:

- Jibson (2007)
- Ambraseys and Srbulov (1995)
- Ambraseys and Menu (1988)
- Bray and Travasarou (2007)

Displacements should be assessed at varying distances from the free face by assessing the yield acceleration for failure planes at increments of 0.5H.

Within close proximity of the reclamation edge, there is a risk of local zones of weaker materials. These materials would be expected to dominate the overall soil strength of the failure plane along which lateral spread could occur.

We propose that a lower bound smeared strength should be adopted for a sensitivity analysis for failure planes within the distance of the free face of two times the depth to the base of the liquefiable material (2H) as presented in Figure 2.

The selection of the lower bound smeared strength requires judgement of the data available from site investigation. For example, the lower bound smeared strength for each depth interval can be selected from one CPT that represents a generally poorer soil profile than the average strength of all CPT tests. It is considered overly conservative to adopt the lowest smeared strength from all CPT/investigation locations.

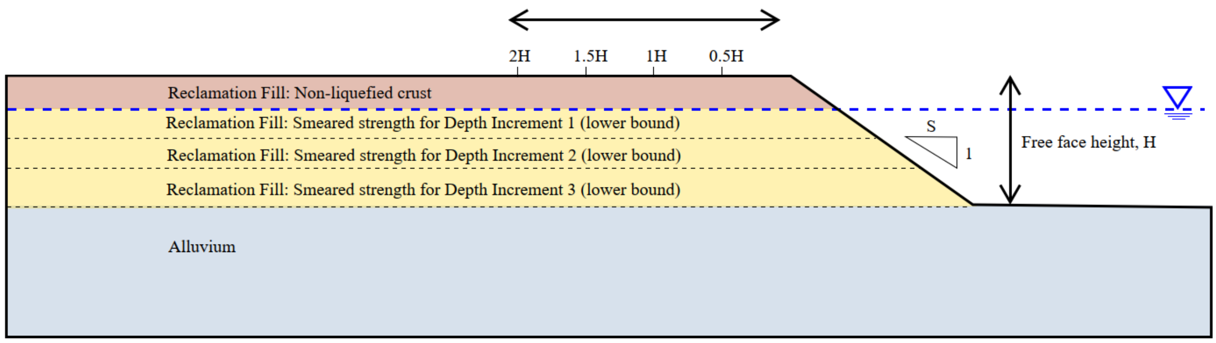

Figure 2: Lower bound 'smeared' strength soil profile and geometry,

5 EXAMPLE SITE

Table 2 summarises the smeared strengths calculated for the ground model presented in Section 2.1. The site is located within flat reclaimed land on the Wellington Waterfront and the free face is the reclamation edge. Three equal depth increments have been considered throughout the reclamation fill, with an average overburden pressure assumed for each depth increment.

Table 2: Design and lower bound 'smeared" strengths calculated

Depth Increment	Range $S_{smeared}$ (kPa)	Design (average) $S_{smeared}$ (kPa)	Lower bound $S_{smeared}$ (kPa)
2 to 5 m (σ'_v = 50 kPa)	14 to 35	22	22
5 to 8 m (σ'_v = 80 kPa)	25 to 38	32	28
8 to 12 m (σ'_v = 110 kPa)	34 to 74	46	40

The results of a NSB assessment, using Jibson (2007) to calculate the seismic displacement is presented in Table 3. The Ambraseys and Menu (1988) and Bray and Travasarou (2007) methods gave results within +/- 30% of those predicted by Jibson (2007). The result predicted by Ambraseys and Srbulov (1995) gave a result of up to twice that predicted by Jibson (2007).

The results are presented for a 500-year return period earthquake in accordance with Module 1 (MBIE/NZGS, 2021) with magnitude M7.7, and peak ground acceleration (PGA) of 0.68g

Table 3: Results of NSB assessment

Soil Strength Profile	Setback from Crest	Yield acceleration, a_y (g)	Seismic Displacement* (mm)
Design	0.5H	0.15	500
	1H	0.13	700
	1.5H	0.14	600
	2H	0.15	500

Soil Strength Profile	Setback from Crest	Yield acceleration, a_y (g)	Seismic Displacement* (mm)
	3H	0.16	500
	5H	0.18	350
Lower Bound	0.5H	0.06	3000
	1H	0.08	1800
	1.5H	0.1	1200
	2H	0.12	800

* Seismic displacement presented for the 16th percentile.

Upon assessment of the results of the NSB assessment, it was proposed to divide the site into two zones as presented in Figure 3. Where:

- Zone A: Outside of 2H of the reclamation edge.
- Zone B: Within 2H of the reclamation edge.

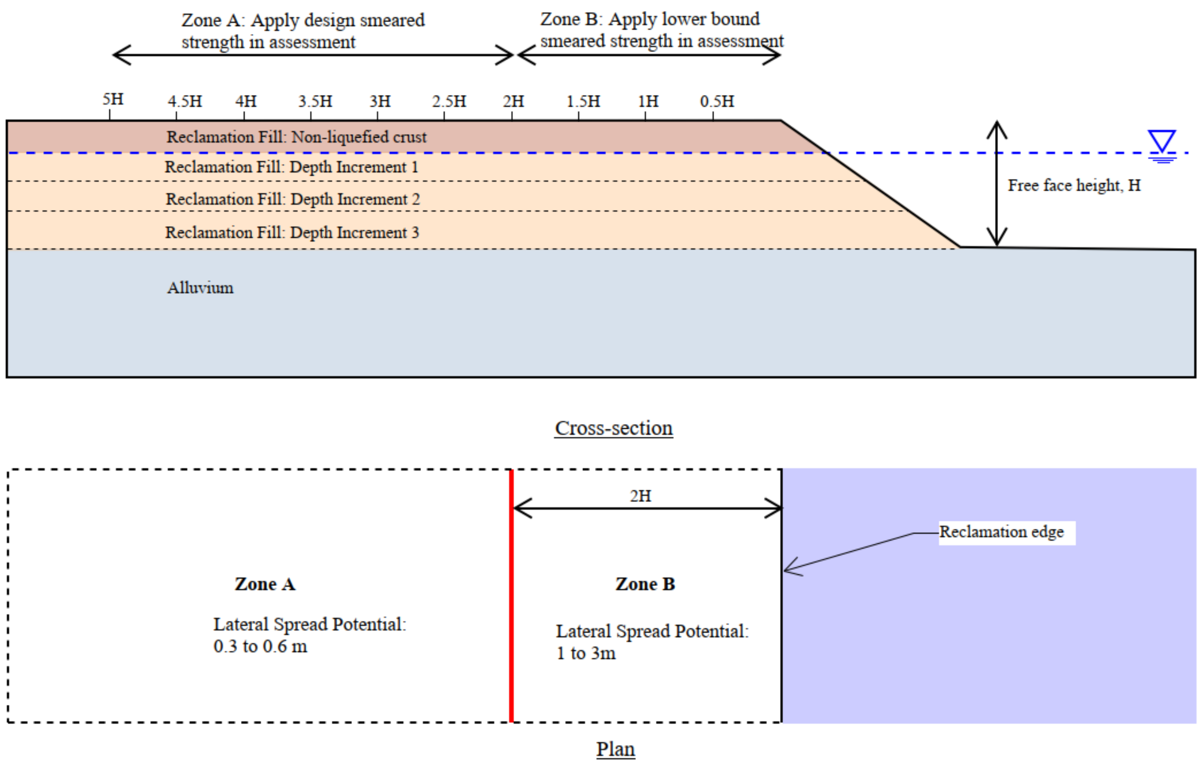

Figure 3: Site zoning

The displacements calculated using the design soil strength profile were applied for Zone A, and the displacements calculated using the lower bound profile for Zone B. The lateral displacements presented should be used as indications of risk to inform future use of the site and any development that may be proposed. It is not recommended to build new structures within Zone B. If structures are to be built within Zone A their design must allow for the possibility of displacements greater than those predicted. Lateral spread displacements cannot be reliably predicted.

6 CONCLUSIONS

Published empirical methods of assessing lateral spread potential are not applicable to highly variable ground conditions where zones of liquefied and non-liquefied soils can be expected. An assessment of lateral spread potential can be made by developing an equivalent 'smeared' strength of the soils and applying this strength in a Newmark Sliding Block type lateral spread assessment. We can then apply engineering judgement to the results of the assessment to identify the potential range of displacements at the site. The uncertainties in these predictions are high and must be allowed for if these predictions are to be applied to a design.

The geotechnical engineer should assess the available geotechnical data and information regarding the method of placement of the fill type when considering if a smeared strength is appropriate for a site. The smeared strength approach should not be used if there is a possibility of a continuous weak layer or weak layers which could be linked as a potential failure surface.

REFERENCES

Ambraseys NN and Menu JM (1988). Earthquake-induced ground displacements. *Earthquake Engineering and Structural Dynamics*, **16**: 985-1006.

Ambraseys NN and Srbulov M (1995). Earthquake induced displacements of slopes. *Soil Dynamics and Earthquake Engineering,* **14**(1): 59-71.

Idriss IM and Boulanger R (2007). Residual shear strength of liquefied soil, *27th USSD Annual Meeting and Conference, Modernization and Optimization of Existing Dams and Reservoirs*, Philadelphia. United States Society of Dams, Denver, Co.

Bray JD and Travasarou T (2007). Simplified procedure for estimating earthquake-induced deviatoric slope displacements. *Journal of Geotechnical and Geoenvironmental Engineering,* **133**(4): 381-392.

Jibson RW (2007). Regression models for estimating co-seismic landslide displacements. *Engineering Geology*, **91**(2-4): 209-218.

Kramer SL and Wang CH (2015). Empirical model for estimation of the residual strength of liquefied soil. *Journal of Geotechnical and Geoenvironmental Engineering*, **141**(9): 04015038.

MBIE and NZGS. 2021. Earthquake geotechnical engineering practice – *Module 1: Overview of the geotechnical guidelines – Version 1.*

Weber JP, Seed RB, Pestana JM, Moss RES, Nweke C, Deger TT and Chowdhury, K (2015). "*Engineering Evaluation of Post-Liquefaction Strength"*. PhD Dissertation, Department of Civil and Environmental Engineering, University of California at Berkeley, 588pp.

Youd, TL, Hansen CM and Bartlett SF (2002). Revised multilinear regression equations for prediction of lateral spread displacement. *Journal of Geotechnical and Geoenvironmental Engineering,* **128**(12): 1007-1017

Zhang G, Robertson PK and Brachman RWI (2004). Estimating liquefaction-induced lateral displacements using the standard penetration test or cone penetration yest. *Journal of Geotechnical and Geoenvironmental Engineering*, **130**(8): 861-871.