A proposed method for estimating differential settlements due to post-liquefaction reconsolidation

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2014 NZSEE Conference

ABSTRACT: Liquefaction-induced differential settlement is challenging to predict because natural variability of the ground is usually not well understood and the complex soil behaviour of liquefaction is difficult to assess. Typical practice is to assume differential settlement due to liquefaction-induced reconsolidation is up to one-half or more of the total predicted settlement, which often leads to large estimated differential settlements that are frequently problematic for typical structures to tolerate. In this study, methodology for estimating ground surface settlement due to tunnel contraction is adapted to estimate differential settlement of the ground surface due to liquefied soils at depth. Liquefaction-induced differential settlement is correlated to the maximum slope of the deformed ground surface by adapting a tunnelling-induced settlement prediction method. The proposed ground surface deformation assessment method is demonstrated in an example problem using both deterministic and probabilistic approaches.

1 INTRODUCTION

Ground surface deformation due to liquefaction is difficult to predict because natural variability of geotechnical properties in the ground is usually not well understood and the complex behaviour of soil liquefaction is difficult to assess. Liquefaction-induced foundation settlement occurs due to several mechanisms (Dashti et al., 2010). Many of the settlement mechanisms identified by Dashti et al. (2010) are reduced or mitigated when the foundation soil is not liquefiable, or has been improved to an adequate depth and extent below the foundation. However, post-liquefaction reconsolidation settlement (reconsolidation settlement), can be attributed to liquefaction occurring at generally greater depth than other settlement mechanisms, and as such, remains a design consideration for engineers.

Typical practice for assessing ground surface deformation due to reconsolidation settlement is to estimate total reconsolidation settlement based on in-situ penetration resistance data (e.g. cone penetration test or standard penetration test), and then assume that differential settlement between foundation elements may be up to half of the total settlement. The conventional "one-half of total settlement" rule-of-thumb has no quantitative basis. This often leads to large estimated differential settlements that may be overly conservative for foundation design.

Differential reconsolidation settlement occurs because of vertical and horizontal variations in geotechnical properties. Typically, geotechnical investigations comprise boreholes or Cone Penetration Tests (CPTs) that represent a small fraction of the area when compared to the size of the project site. Designers use their judgement to assess how the variability in ground conditions influences the reconsolidation settlement and associated ground surface deformation. In addition, confidence in ground characterisation and available liquefaction assessment methods contributes to the overall uncertainty in design. Thus, "one-half of total settlement" is a rule-of-thumb to account for both natural variability and engineering uncertainties.

Natural variability in the context of post-liquefaction settlement comprises a) the geometry of the liquefiable strata, and b) variations in gradation across a site. For example, various geologic conditions and their expected ground surface settlement profiles are summarised schematically on Figure 1.

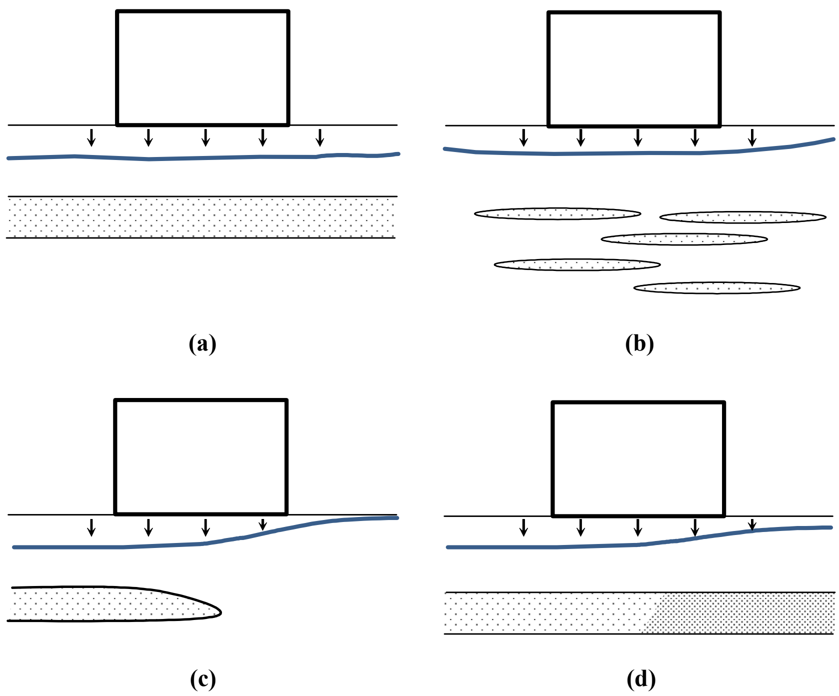

Figure 1. Example settlement profiles for various subsurface conditions.

The critical layer shown on Frame A is laterally continuous and has uniform gradation. Total reconsolidation settlement may be relatively large; however settlement is expected to occur uniformly across the site. This may manifest as a global or regional subsidence.

Frame B depicts a highly interbedded geology comprising multiple thin layers of potentially liquefiable soils. An accurate understanding of the geometry of this condition is impossible to characterise; however, a typical geotechnical investigation could assess the degree and uniformity of the interbedded layers. The aggregate contribution of the interbedded layers to the ground surface deformation is expected to result in generally uniform settlement that is commensurate with the frequency and thickness of liquefiable layers. Similar to Frame A, this may manifest as a global or regional subsidence.

Frame C, in contrast, depicts a condition where the potentially liquefiable soil layer is present only below a portion of the site. It is apparent that this condition has potential for large differential ground deformation. This condition should be readily identified and characterised by a typical geotechnical investigation, which will allow a design that is appropriate for the variable ground conditions.

Frame D depicts a condition where the critical layer is laterally continuous below the site, but varies in gradation or density. This condition, similar to Frame C, poses an obvious risk for differential settlement, and should be characterised by a typical geotechnical investigation.

The ground is seldom as uniform or simply represented as depicted on Figure 1. This is evident by comparison of any two boreholes or CPTs conducted on a given "uniform" site, such as the site shown on Frame A. The soil within the critical layer is similar in age and deposition, but the data from the two boreholes or CPTs will always be different, even if just slightly different. The same is true considering three boreholes or CPTs, and so on. Therefore, a better depiction of a liquefiable strata may be similar to Frame D, but with multiple changes in gradation and density (i.e., the natural variability).

In addition to site conditions, there are also uncertainties in reconsolidation settlement prediction associated with analysis methods for assessment of liquefaction potential and liquefaction-induced volumetric strain. It is typically not cost effective to develop a thorough understanding of alleatory variability (natural variability of the ground conditions) and epistemic uncertainty (uncertainty associated with our analysis and design methods), so geotechnical engineers often treat variability and uncertainty simply by characterising a site based on average or conservative settlement estimates using rules-of-thumb and/or large factors of safety to estimate ground surface deformation for geotechnical design.

The methodology proposed in this study provides a quantitative means to account for the effect of a non-liquefied crust on propagation of differential reconsolidation settlement. Ground surface deformation due to reconsolidation settlement is proposed to be assessed by use of principals developed to estimate ground surface settlement due to tunnel construction.

2 METHODOLOGY

The proposed methodology for assessment of reconsolidation-induced ground surface deformation assumes that post-liquefaction volumetric strains within a liquefiable soil layer propagate to the through a non-liquefied crust to the ground surface in a similar manner that tunnelling induced deformations propagate to the ground surface. Ground surface deformation due to tunnel excavation is related to a number of factors, including the geotechnical conditions around the tunnel, tunnel excavation method, tunnel lining design, and construction practices. Tunnel excavation causes vertical and horizontal deformations and stress redistribution in the ground around the tunnel. This behaviour results in arching, or bridging, of the ground as it accommodates the loss at the tunnel face and relaxation of the tunnel lining. The resulting ground surface settlement profile was first characterised by Peck (1969) as an inverted normal distribution based on empirical relationships. Since 1969 numerous methodologies and design procedures have been proposed to estimate the shape of the settlement profile.

Reconsolidation settlement is typically estimated as the cumulative settlement due to post-liquefaction volumetric reconsolidation strains in potentially liquefiable soil layers below a site (e.g., Tokimatsu and Seed, 1984, Ishihara and Yoshimine, 1992, Shamoto et al., 1998, Zhang et al., 2002, and Cetin et al., 2009). The available analysis methods were developed for the case of one dimensional reconsolidation, and as such the fundamental assumption of these methods is that the ground surface is flat, or nearly flat, and the critical layers are laterally continuous. However, where volumetric reconsolidation settlement varies over areas of limited extent due to changes in grain size, gradation and density, such as shown on Figure 2, the non-liquefied crust material can bridge over the liquefied soil to disseminate the settlement as it propagates to the surface. This is conceptually similar to the effect of overburden soil on tunnelling-induced deformation.

Logonathan and Poulos (1998) parameterise tunnelling-induced ground settlement using four values, the depth of the tunnel springline, H, the radius of the tunnel, R, Poisson's ratio of the ground around and above the tunnel, ν, and the gap parameter, g, as shown on Figure 3. The gap parameter proposed by Logonathan and Poulos (1998) is the maximum displacement of an ovalised tunnel lining and depends on tunnel design, construction method, and workmanship. Logonathan and Poulos (1998) compute the ground surface settlement, $U_{z=0}$, using Equation 1:

$$U_{z=0} = 4(1-\nu)R^2 \frac{H}{H^2+x^2} \frac{4gR+g^2}{R^2} \exp\left[-\frac{1.38x^2}{(H+R)^2}\right] \quad (1)$$

where x is the lateral distance from the tunnel centreline.

The point of inflection of the inverted normal distribution, or trough width, i, is computed using Equation 2:

$$\frac{i}{R} = 1.15\left(\frac{H}{2R}\right)^{0.9} \quad (2)$$

In this paper it is proposed that these same methods may be used to estimate the maximum ground surface deformation, or ground slope, B, (B horizontal:1 vertical) due to variable reconsolidation settlement occurring within a liquefied strata. In order to use the relationships described by Logonathan and Poulos (1998) to estimate ground surface deformation due to reconsolidation settlement the gap parameter (g) is re-defined as gap settlement, S_{gap} as shown on Figure 3. S_{gap} is the differential settlement expected for a given liquefied soil strata and is the difference between a local maximum settlement, S_{max}, and the global settlement. S_{gap}, S_{max}, and S_{global} can be estimated by any available method for determination of reconsolidation settlement along with judgement based on the number and type of intrusive investigations and other geotechnical data. For most applications S_{global} could be the average settlement or a local minimum settlement expected for the site; whereas S_{max} could be selected to represent a greater settlement depending on the range of ground conditions (i.e. S_{max} is a local maximum settlement, not an absolute maximum). The orientation or exact location of changes in gradation and density do not need to be defined in order to estimate limiting values of B.

The depth and radius parameters used by Logonathan and Poulos (1998) can be written in terms of thickness of non-liquefied crust, d, and thickness of liquefiable soil, t respectively. Table 1 provides a summary of the parameters for reference.

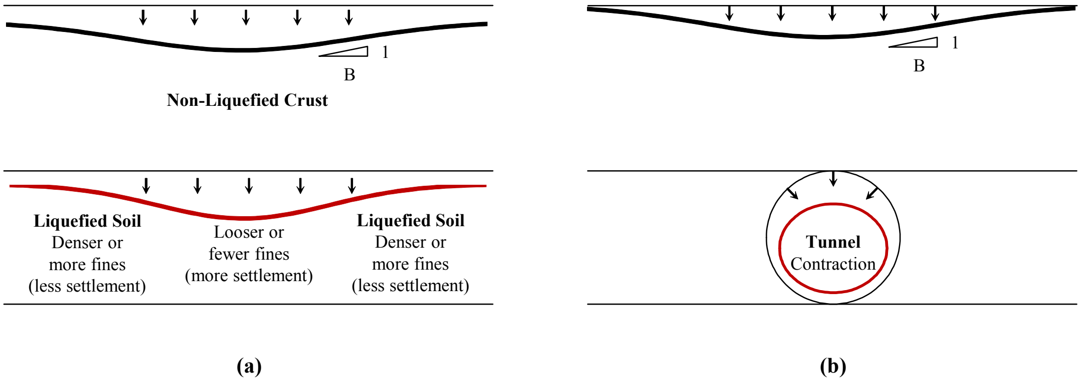

Figure 2. Settlement profiles due to (a) reconsolidation settlement in variable ground conditions and (b) tunnel contraction.

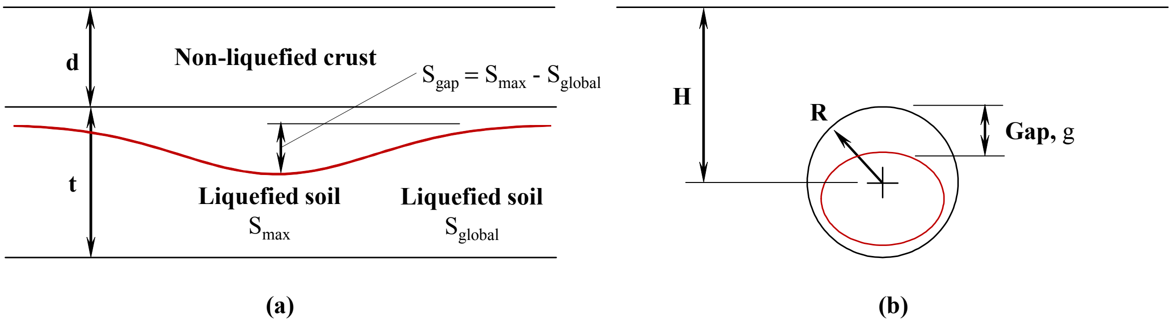

Figure 3. Parameters for estimating ground surface deformation due to (a) reconsolidation settlement and (b) tunnel construction.

Table 1. Summary of input parameters.

Parameter	Logonathan and Poulos (1998)	Reconsolidation-induced ground surface deformation
Depth of tunnel springline	H	d + t / 2
Radius of tunnel	R	t / 2
Gap parameter/ differential settlement	g	S_{gap}
Poisson's ratio	ν	ν

3 PROPOSED DESIGN METHOD AND EXAMPLE PROBLEM

The example project site is located in Sydenham approximately 2 km south of Christchurch CBD. The site was occupied by an aged care facility prior to the 2010-2011 Canterbury Earthquakes. The previous structures were supported on unreinforced concrete slab foundations that suffered extreme damage due to liquefaction. The aged care facility was demolished after the earthquakes, and a new development of 31 retirement villas with a hospital facility is proposed. This example is focused on design of the hospital facility. The foundation design for the proposed hospital facility comprises a reinforced concrete slab supported on ground improved to a depth of 10 m. The reinforced concrete slab was designed to tolerate a maximum post-liquefaction ground surface deformation of 300:1 (H:V).

The preliminary design of the hospital facility is based on the geotechnical data available from the geotechnical investigation for the retirement villas, which comprises 36 CPTs that extend through about 25 m of bedded sediments to terminate in the dense gravels that underlie Christchurch (Frame A of Figure 4). The geotechnical investigation encountered potentially liquefiable silt and sand throughout the soil profile to a depth of about 20 m. The soils near the ground surface in Sydenham are highly variable in grain size, gradation and density, having been subject to, and deposited by, a variety of geomorphic processes. The soil between a depth of about 8 m to 20 m were deposited during a period of sea level rise, and are relatively consistent compared to the near-surface soils. The vertical and lateral variability in the geologic units is demonstrated through comparison of CPT tip resistance, q_t, (Figure 4). The CPT locations (Frame A of Figure 4) are colour coded based on cumulative settlement index (i.e. the directly computed value) below the 10 m deep improved zone. The cumulative settlement index ranges from 80 mm to 190 mm and has a mean of 130 mm. The site geology below a depth of about 10 m is consistent between the aged care and hospital facilities, so the data collected for the aged care facility is useful for assessing the preliminary design of the hospital (i.e. ground conditions need to be confirmed on site for detailed design).

The design depth of ground improvement was selected as 10 m in order to reduce total reconsolidation settlement and increase the bearing strength of the foundation soils. Assuming that liquefaction hazards and bearing capacity are satisfied by the proposed ground improvement, the following examples will demonstrate how the methodology proposed in this study could be used to assess the ground surface deformation.

A typical approach for estimating differential settlement is to assume that up to half of total settlement could occur as differential settlement between footings or columns. Following the typical approach, differential settlement could be estimated to be up to 95 mm, or one-half of the maximum calculated total settlement index of 190 mm. This magnitude of differential settlement may be does not meet the design criteria for a typical footing or column spacing in the order of 8 m to 10 m. Instead, the methodology proposed in this paper may be used to estimate the benefit of the non-liquefied crust by entering values for a 10 m crust, 10 m liquefiable layer thickness, and S_{gap} of 95 mm into Logonathan and Poulos (1998). The estimate for natural (unimproved) ground conditions with in-situ lateral stress coefficient, K_0, of 0.4 yields a maximum ground surface deformation of about 164:1 (H:V) (i.e. B = 164), and slope of 460:1 (i.e. B = 460) if the ground is improved to provide average K_0 of 2.

The use of settlement index values does not consider alleatory variability or epistemic uncertainty, so it is common to allow a factor on the computed settlement to provide a safety margin on foundation performance. This factor is selected based on engineering judgement to account for both types of uncertainty, and conservatism in the estimated total settlement, which is then passed through to the estimated differential settlement.

The proposed method for estimation of ground surface deformation due to reconsolidation settlement reduces epistemic uncertainty because the method relies on assessment of relative differences in computed total settlement, as opposed to obtaining an accurate estimate of total settlement. In this regard, the method is not sensitive to total settlement because any combination of liquefaction assessment method (e.g., Youd et al., 2001, Robertson and Wride, 1998, Cetin et al., 2004, Moss et al., 2006, Idriss and Boulanger, 2008) and settlement evaluation method (e.g., Tokimatsu and Seed, 1984, Ishihara and Yoshimine, 1992, Shamoto et al., 1998, Zhang et al., 2002, and Cetin et al., 2009) is expected to provide a similar range of the estimated settlement. Epistemic uncertainty could be reduced further by considering multiple analytical methods.

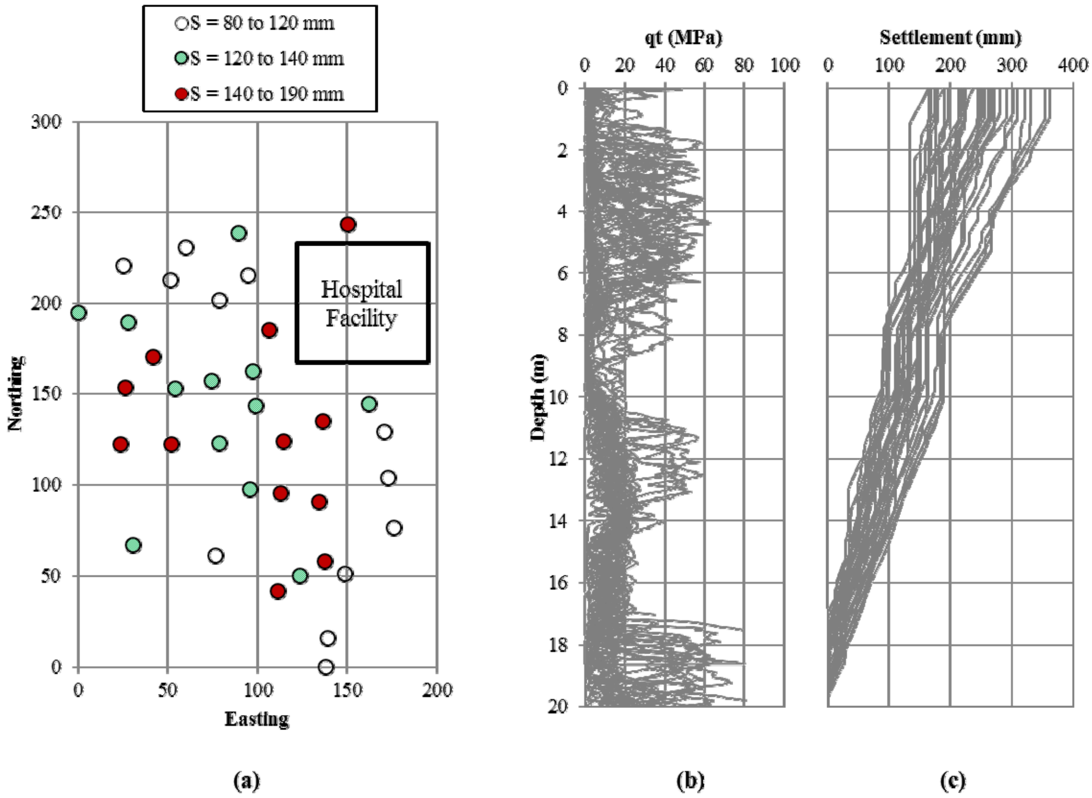

Figure 4. Spatial and statistical distribution of settlement due to reconsolidation of liquefied soil below depth of 10 m (a) CPT location plan (units of axes are meters); (b) q_t profiles; and (c) cumulative reconsolidation settlement index profiles.

Alleatory variability can be assessed using a probabilistic approach to evaluate the ground surface deformation for the project. For example, the large number of CPTs available for the example problem can be used to provide a better understanding of ground surface deformation by considering the probability of exceedance of a post-liquefaction ground slope, p(B). There are three steps required to calculate p(B):

1) Describe the natural variability in ground conditions using a probability distribution of total PLCS, S;
2) Compute the probability of S_{gap} from the total PLCS distribution;
3) Map ground surface slope to S_{gap} with the method proposed in this paper.

Following these steps p(B) can calculated from Equation 3 as:

$$p(B) = \mathrm{p}(S_{gap} > x) = \mathrm{p}(S) \cdot \mathrm{p}(S_{gap} > x|S) \tag{3}$$

where $p(S_{gap} > x)$ is the probability of exceedance of S_{gap} between any two foundation elements; p(S) is the probability of S at any location below the hospital; and $p(S_{gap} > x|S)$ is the probability of exceedance of S_{gap} between any two foundation elements given an initial value of S.

Frame A of Figure 5 depicts a histogram of the index settlement, S, computed for the 36 CPTs. The histogram is fit by a lognormal distribution of the probability of S, p(S). This distribution represents the likelihood of index settlement at any given location below the proposed hospital facility. This is a reasonable assumption because the depositional environment of the soil beneath the depth of 10 m was consistent below both the retirement villas and hospital facility portions of the site.

Using p(S), the probability that S_{gap} between any two foundation elements exceeds a value of x, $p(S_{gap} > x|S)$, is calculated as the area below the portions of the total settlement distribution that are outside the bounds of S ± S_{gap}, an example of which is shown on Figure 5. This approach conservatively assumes that settlement between any two locations is random and uncorrelated.

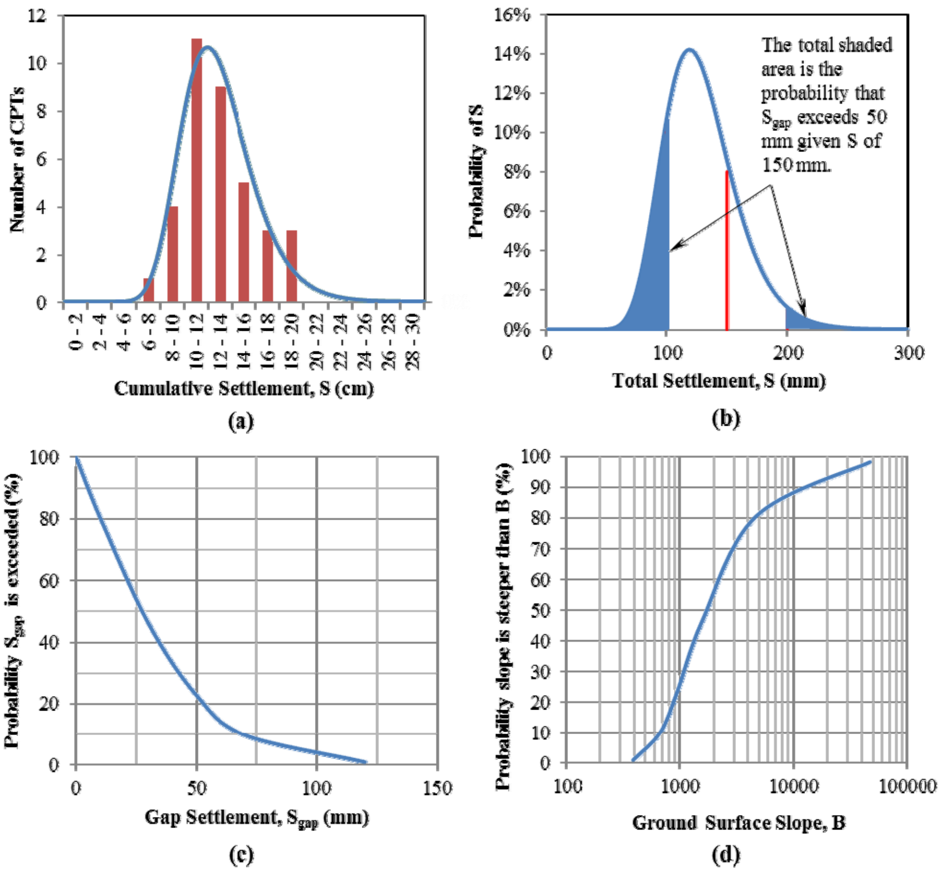

Figure 5. Calculation of p(B): (a) settlement histogram; (b) probability of gap settlement given total settlement, p(S_{gap}|S); (c) probability of gap settlement, p(S_{gap}); and (d) probability of slope, p(B).

Lastly, the post-liquefaction ground slope, B, is calculated using the methodology proposed in this paper, and then p(B) is determined as the corresponding p(S_{gap}). As shown on Figure 5, the probability that the ground surface deformation exceeds the design threshold B=300 is negligible for the example problem. The low probability is due to the fact that the range of S is small compared to the S_{gap} required to produce a ground slope steeper than 300:1. This conclusion is not attainable by typical, or deterministic, analyses. The analyses described in this paper rely on a deterministic assessment of cumulative total reconsolidation settlement to develop a probabilistic model of S; thus consideration of probabilistic methods in the assessment of liquefaction potential and post-liquefaction volumetric strain could provide a further improvement in assessing ground surface deformation.

4 CASE HISTORY AND NUMERICAL ANALYSIS VALIDATION

Validation of the proposed method for estimating ground surface deformation using LiDAR data and CPTs collected at sites across Christchurch after the 2010-2011 Canterbury Earthquake Sequence is in progress, but is not yet complete at the time of writing this paper. In addition, a suite of numerical models to assess the benefit of the non-liquefied crust on reducing ground surface deformation have been conducted, but likewise are not yet complete at the time of writing this paper.

5 CONCLUSION

A typical method for estimating ground surface deformation due to post-liquefaction reconsolidation uses the rule-of-thumb that differential settlement may be up to one-half of total reconsolidation settlement. This approach does not account for the benefit of overlying non-liquefiable materials on dissemination of the settlement through the non-liquefied crust. This study overcomes this limitation and proposes a method to estimate ground surface deformation associated with reconsolidation settlement that is based on the method of Logonathan and Poulos (1998) to estimate ground surface deformation associated with tunnelling-induced settlement. The method has not yet been fully validated, but the logic is sound and has been used to inform preliminary assessment of likely ground surface deformation for an important structure in Christchurch.

The proposed method can be applied without knowledge of location or orientation of variable ground conditions to provide a quantitative estimate of differential settlement, which can be easily extended to consider the probability of ground surface slope, p(B). The use of probabilistic analyses can assess the sensitivity of B to S_{gap}, and in doing so, can reduce uncertainty in engineering judgement.

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