Assessment of liquefaction induced settlement within engineering practice

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ABSTRACT

The assessment of the magnitude of liquefaction-induced settlements affecting shallow founded buildings and buried services is an important yet challenging task for geotechnical engineers. It is often required when evaluating foundation options for a proposed new development, where the potential economic benefit of shallow foundations, sometimes with ground improvement, must be assessed against deep foundation options that may have a lower vulnerability to liquefaction induced settlements. It is also often required for detailed seismic assessments of existing structures founded on liquefiable ground. While recognising the significant uncertainties in the derived estimates, a considered application of published analytical methods for quantifying liquefaction induced building settlements (e.g. Bray & Macedo, 2017; Bullock et al., 2019), can be useful to develop an understanding of anticipated performance. This paper summarises assessment requirements and provides pragmatic advice for determining appropriate ground motion parameters for the new methods, and interpretation of results drawing on recent publications, observations from earthquakes, and the authors experience in applying these considerations in engineering practice.

1 INTRODUCTION

A quantitative assessment of liquefaction-induced ground settlements is important for the design of new-build structures, and the assessment of existing structures where there is a risk of liquefaction within underlying soils. Foundation movements from liquefaction-induced settlement can be significant, causing deformations to structures that exceed the serviceability limit state (SLS) and/or ultimate limit state (ULS) requirements in New Zealand design standards. The assessment can be quite complex, requiring the geotechnical engineer to assess the liquefaction hazard, foundation stability following any anticipated strength and stiffness loss, and quantify the ground deformation – total and differential settlement and possibly lateral ground movements. The significance of anticipated ground movement needs to be interpreted in conjunction with the structural engineer (for buildings), mechanical or civil engineer (for infrastructure, e.g. pipelines, plant) to assess the consequences.

Significant research work has been undertaken in recent years to develop prediction equations that more fully capture the complex mechanisms that contribute to building settlement on liquefiable soils, specifically those presented by Bray & Macedo (2017) [BM17] and Bullock et al. (2019) [BEA19]. Where appropriate these models may be adopted by engineering practitioners to better inform engineering judgement in the assessment of foundation performance in earthquakes. This paper presents a review of settlement estimates including these new models, with consideration for application in New Zealand engineering practice.

1.1 Assessment within the NZ regulatory context

The NZGS/MBIE (2016) guidelines for Earthquake Resistant Foundation Design summarise the regulatory requirements in New Zealand. In short, compliance with the NZ Building Code is achieved through conformance with the B1 verification methods, or appropriate 'alternative solutions'. Structural design actions (per AS/NZS 1170.0:2002) are applied to foundation elements, which must be appropriately sized to resist those loads. NZS1170.5:2004 requires that ULS deformations during or immediately following an earthquake be limited so that the structural system continues to safely perform load bearing, and that non-structural systems do not inhibit emergency evacuation of the building. For foundations this would limit the permissible total and differential settlements, and is dependent on the particular structure's tolerance, as assessed by the structural engineer. The SLS deformations are limited to being essentially negligible (no or minor cosmetic repairs).

1.2 Detailed Seismic Assessments (DSA) of existing structures

Section C4 of the 'NZSEE guidelines' (NZSEE et al. 2017) presents the process to determine whether geotechnical issues may impact the *%NBS* rating (percent of New Build Standard), by which the earthquake 'prone' or 'at risk' categories are determined, which have implications under the Building Act 2004 and associated legislation related to earthquake prone buildings. Soil-foundation deformations that affect the building structure are among the range of potential geohazards considered in a DSA. Specific consideration is made to so called 'step change' behaviour, where a non-ductile foundation response may occur, defined as a sudden loss of building support once a threshold level of shaking is exceeded. Shallow foundations founded within a relatively thin non-liquefied crust, or directly on liquefiable soils may be subject to a punching-shear type failure, or a progressive failure due to a cyclic ratcheting response facilitated by a combination of building shaking movement and cyclic softening of founding soils, resulting in potentially large differential settlements. Such movements may be well tolerated by robust structures, but not others which have pre-existing structural weaknesses that may potentially affect the life safety objectives at the ULS, and hence the *%NBS* rating. It is this potential for structural weaknesses to be exacerbated by liquefaction-induced ground movements that may require an 'interactive' assessment between structural and geotechnical engineers in order to assess the *%NBS*. Note that on sloping ground, liquefied soils may facilitate large ground movements and extreme deformations of structures, would likely be considered a 'geotechnically dominated' assessment if this risk is present, and is a separate case to the level-ground settlement prediction methods considered in this paper.

2 MECHANISMS OF BUILDING SETTLEMENT

Seismically induced settlement of level ground is the result of a combination of complex mechanisms that occur to the soil and structure during and post-shaking which contribute to the final settlement of the structure. A diagrammatic representation of these interactive mechanisms is presented in Figure 2-1 (after Bray & Dashti, 2014). In summary three components are considered to contribute to seismic induced building settlements:

- Volumetric-induced ('free field' settlement), D_v:

  Includes the sedimentation and re-consolidation of liquefied soils post-shaking as well as any 'shake down' settlement of loose dry granular soils and poorly compacted fills (Figure 2-1 d & e). These settlements occur independent of the presence of any overlying structure.

- Shear-induced, D_s:

  The presence of buildings may cause further settlement due to additional shear strains induced in founding soils by the building loads, which are significantly impacted by the dynamic response of the building and its interaction with soil during strong shaking (i.e. rocking). This will be worsened by the development of liquefaction or cyclic softening within a zone of influence of the foundation where it may lead to the development of a 'punching shear' type bearing failure (Figure 2-1b), or the progressive mechanism referred to as 'SSI-ratcheting' (Figure 2-1c). Overall building settlements are more significant for taller, heavier buildings than for shorter, lighter buildings (Anderson et al. 2007; Bray & Dashti 2014; Bray & Macedo 2017).

- Ejecta related, D_e:

  Further settlements may occur from loss of soil from below foundations, intensified by oscillating stresses imposed by the structure causing the pumping of liquefiable soils beneath. This action leads to high excess pore water pressures, and steep hydraulic gradients developing beneath the foundations which drive the dissipation outwards and upwards during and post shaking. This flow of pressurised pore fluid may further fluidise and/or entrain sand and fines, bringing it to the ground surface, to be deposited as sand ejecta (i.e. sand boils, Figure 2-1a) or by redeposition within poorly graded soils e.g. open graded hardfill rafts. The movement of foundations during shaking also damages the surface 'crust' of dry or otherwise non-liquefied soil, facilitating localised venting of pressurised ground water, and after major earthquakes it is common to observe deposits of sand ejecta around the perimeter of buildings where liquefaction has occurred beneath the foundation.

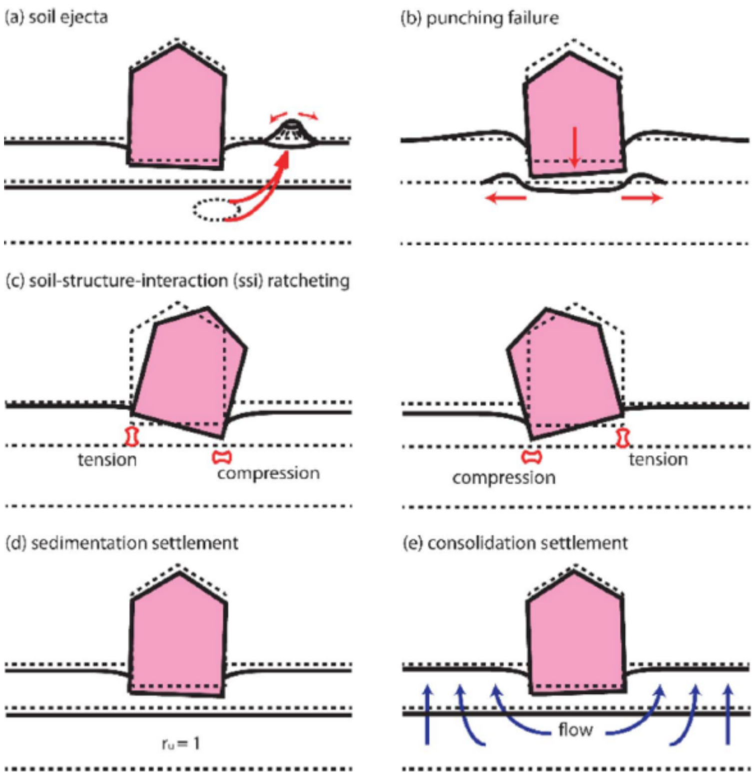

Figure 2-1: Liquefaction induced building displacement mechanisms: (a) ground loss due to sand ejecta; shear-induced settlement from (b) punching failure, or (c) soil-structure interaction (SSI) ratcheting; and volumetric induced settlement from (d) sedimentation or (e) post-liquefaction reconsolidation (from Bray & Macedo 2017, modified from Bray & Dashti 2014).

2.1 Free field settlements, D_v

Component D_v includes both the 'shake-down' of dry granular soils, and saturated soils that do not liquefy, as well as liquefaction-induced settlements, with the latter being most significant in terms of magnitude. For large thicknesses of liquefied soil in the field, total reconsolidation settlements may be significant (100's of mm, up to 1m+).

2.1.1 Semi-empirical methods

The CPT-based methods presented by Zhang et al. (2002), Idriss & Boulanger (2008), as implemented in industry software such as *CLiq*, or that of Yoshimine et al. (2006) as implemented in Settle3 software are perhaps most commonly applied by practitioners. All approaches rely on relationships presented by Ishihara

& Yoshimine (1992) between the Factor of Safety against liquefaction triggering, FS_{Liq} (as determined from the simplified assessment method), and the cyclic shear strain, γ, and volumetric strain, ε_v of sand, from laboratory cyclic test results for a clean sand (Toyoura) of varying density (refer Figure 2-2 A).

Tokimatsu & Seed (1987) present a commonly adopted simplified method using SPT *N* data, for both shake-down and liquefaction-induced settlements, through relationships between ε_v and cyclic shear stress ratio *CSR*, varying with stress normalised $(N_1)_{60}$ value (refer Figure 2-2 B). A variant of the approach is presented by Cetin et al. (2009), and Wu (2002) also provides charts based on SPT *N* value. The relationships in all cases are based on lab testing of clean sands.

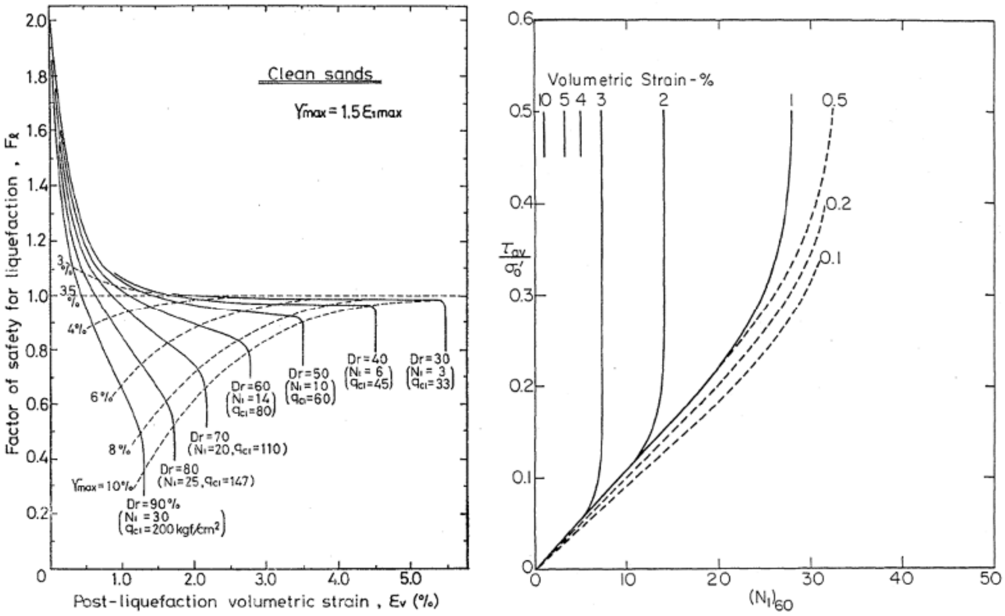

Figure 2-2: A) Factor of safety to volumetric strain relationship for varying sand density (Ishihara & Yoshimine, 1992), and B) Chart to determine volumetric strain from cyclic stress ratio and stress-normalised SPT N value (Tokimatsu & Seed, 1987).

Key differences between these approaches lie in the adopted CPT- or SPT- correlation to relative density, D_R, and a further consideration is how fines (silt & clay fraction) are considered. In application to soil profiles that contain silts it is assumed that the fines content (FC) corrections adopted for assessing liquefaction triggering are sufficient to also address any differences in the calculation of reconsolidation settlements, though this is not clearly established, and studies are limited (NASEM, 2016). Stewart & Whang (2003) refer to cyclic test results on saturated sands with fines showing approx. ¼ the ε_v at 15 cycles than clean sands of the same D_R, suggesting the FC corrections for liquefaction triggering are more than likely under-correcting for the difference in behaviour. It is also known for example that vibration-induced ground improvement techniques (vibroflot/ stone columns) are much less effective at inducing densification in sands containing significant silt, and it may be inferred that earthquake-induced shaking will similarly be less effective at densifying silty sands.

2.1.2 Numerical analysis

Sophisticated Non-Linear dynamic effective stress Analyses (NLA) may be employed to model seismic site response including post-liquefaction settlements. This type of analysis is described in general by Cubrinosvki (2011) with the main steps presented in Figure 3. Some recent studies which incorporate this approach to evaluate free-field liquefaction and associated effects include Cubrinovski et al. (2019); Hutabarat & Bray (2021), and Montgomery & Boulanger (2016), while Pyke (2020) describes an approach to calibration using published test data for the purposes of free-field liquefaction-induced settlement prediction. The basis for the liquefaction induced building settlement prediction models presented later in this paper (Bray & Macedo 2017; Bullock et al. 2019) are based on parametric studies using NLA.

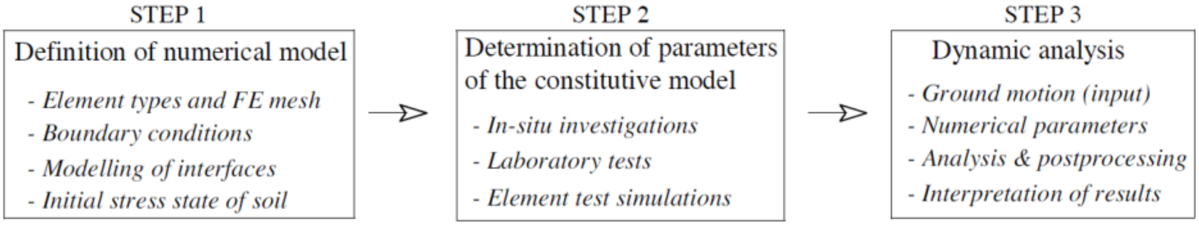

Figure 3: Main steps/ procedures in seismic effective stress non-linear analysis (NLA) (Cubrinovski 2011).

2.1.3 Recognised limitations

These simplified methods typically have poor accuracy and often result in significant over-estimation compared to field observations (Cetin et al. 2009; Geyin & Maurer 2019; Pyke 2020). Geyin & Maurer (2019) reviewed more than 1000 case histories from the Canterbury earthquakes and present comparisons of LiDAR based measurements of D_v with estimates using CPT-based procedures, with the error showing a strong bias with magnitude of predicted settlement (refer Figure 2-4), with low estimates under-predicted and D_v > 60mm typically over-predicted, and a fairly large degree of scatter (34% data points ± 100mm, 10% ± 200mm). It is not clear whether the study used pre-quake CPT results for the evaluation. If post-quake CPT results were used, these may not be an entirely fair assessment of the procedure as they would be affected by changes brought about by earthquake shaking and the associated liquefaction (the degree to which is unknown). The observed trend however may be on account of soil layers at greater depths either not triggering (but predicted to do so), or not contributing significantly to settlement observed at the ground surface. Furthermore, we note the following:

- The assumption that, all other things being equal, both shallow and deep liquefied layers contribute equally to D_v is inherent to these simplified approaches, and there are a number of reasons why this is unlikely to be true in reality, e.g. lateral inhomogeneity of subsurface stratigraphy (i.e. discontinuous "pockets" of liquefaction) and soil arching between non-liquefied zones (Cetin et al. 2009; Geyin & Maurer 2019; Pyke 2020).
- Sand layers with significant fines content, as noted earlier, are likely to be over-estimated. Recall that due to grain size differences, a sandy soil with > ~30% fines will tend to be dominated by the silt grain contacts, not those of sand (Cubrinovski & Ishihara, 2002).
- CPT-data in thinly interbedded sand and clay deposits under-estimates the resistance in sandy layers <1m thick, and consequently over-estimating liquefaction triggering (lower FS_{Liq}), leading to higher volumetric strains and resulting settlement predictions (refer Boulanger et al. 2016).
- D_v predictions rely on the simplified liquefaction triggering procedure which does not consider the build-up of excess pore pressures or its dissipation with time or the interaction between layers within a soil profile, or lateral spatial variations; or the effects of cyclic soil softening on ground motion propagation. When triggering occurs, the subsequent passage of earthquake ground motion is significantly damped, and this may limit the extent of triggering within a soil profile once triggering has occurred in a deeper layer.
- Note that D_v estimates do not consider the impact of structures on the settlement predictions but are an important 'index' for expected ground damage, and feed into some building settlement assessment methods discussed subsequently.

Some of the effects discussed above can only be considered in more sophisticated NLA. However, NLA is currently rarely used by engineering practitioners as the approach is considered by many to be time consuming, costly, and demanding on the knowledge of the modeller (and peer reviewers). Furthermore, comparisons to physical model testing and well documented case histories shows that accuracy of NLA settlement predictions are highly dependent on the constitutive model used and its calibration (e.g. Ramirez et al. 2018; Kutter et al. 2020). Consequently, and despite their limitations, reliance in engineering practice is still largely placed on simplified approaches. For important or sensitive structures with a preferred shallow foundation option, where settlement predictions form part of the design basis, advanced modelling should be considered, and the additional effort and cost factored into design programmes and budgets.

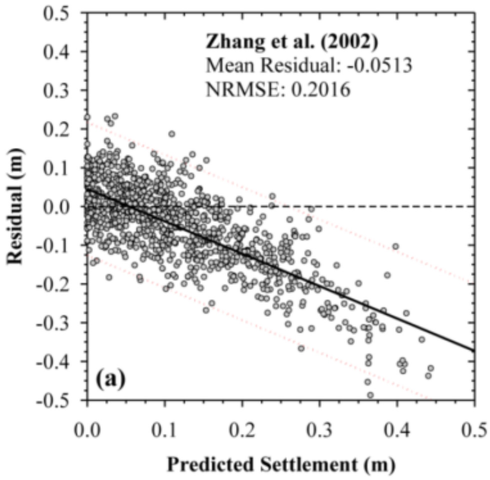

Figure 2-4: Predicted free-field settlement using the CPT-based Zhang et al. (2002) method vs. residual errors (with respect to observed settlement following the 22 February 2011 Christchurch earthquake), with linear regression line and 95% prediction bands. Geyin & Maurer (2019).

2.2 Shear-induced settlements, D_s

2.2.1 Historical approaches

It has long been realised that further settlements will result from the shear stress owing to building loads compared to the free field, and that these are a function of the width and contact pressure of the foundation and thickness of the liquefied soil layer, among other factors (Yoshimi & Tokimatsu, 1977). Ishihara & Tokimatsu (1988) presented a simplified equation that considered total settlements arising from a building footing placed on a two-layer soil (non-liquefied crust over liquefied soil), to be a combination of volumetric settlement, D_v, and immediate settlement, D_c, as a result of a reduction in soil stiffness, requiring the stiffness of the two layers, footing width B, and bearing pressure q, as input. Alternatives that have been adopted in practice, involve analytical or numerical methods to model the two-layer static settlement problem such as elastic settlement analyses (i.e. Boussinesq or similar, e.g. Settle3 software) or numerical analysis (e.g. Plaxis, FLAC software etc.).

Liu & Dobry (1997) from centrifuge scale model testing, recognised that building settlements were also strongly influenced by inertia forces on the structure. They presented an empirical chart based on field observations from earthquakes that correlates the width of the building, B, normalised by the thickness of liquefaction D_L, to the magnitude of foundation settlements also normalised by D_L (refer Figure 2-5 A). However, a large range of building settlement predictions is implied by the chart, reducing its practical use for engineering assessments. Additional physical scale model/ centrifuge studies in the past decade have also highlighted the limitations of the chart and indicated some mechanisms of shear-induced settlement that were not captured by the curves (Bray & Dashti 2010; Adamidis & Madabhushi 2017, refer Figure 2-5 B).

2.2.2 Numerical analysis

Dynamic soil-structure interaction performed using NLA, using an appropriately calibrated advanced soil constitutive model, can effectively capture the majority of key features observed in experiments and well documented case histories (e.g. Dashti & Bray 2013; Karimi & Dashti 2017; Macedo & Bray 2017; Bray & Luque 2017). While able to capture D_s relatively well, they are not able to capture D_v as well, and D_e not at all. As noted previously, NLA is demanding and may not be suited to many projects in practice but should be considered where the performance of sensitive or important structures are being evaluated. Murashev et al. (2015) present an example of such methods being applied to a shallow foundation problem in NZ engineering practice to obtain liquefaction-induced building settlement. Such tools have been used by researchers to undertake parametric studies to inform new simplified settlement and tilt prediction models discussed subsequently in this paper.

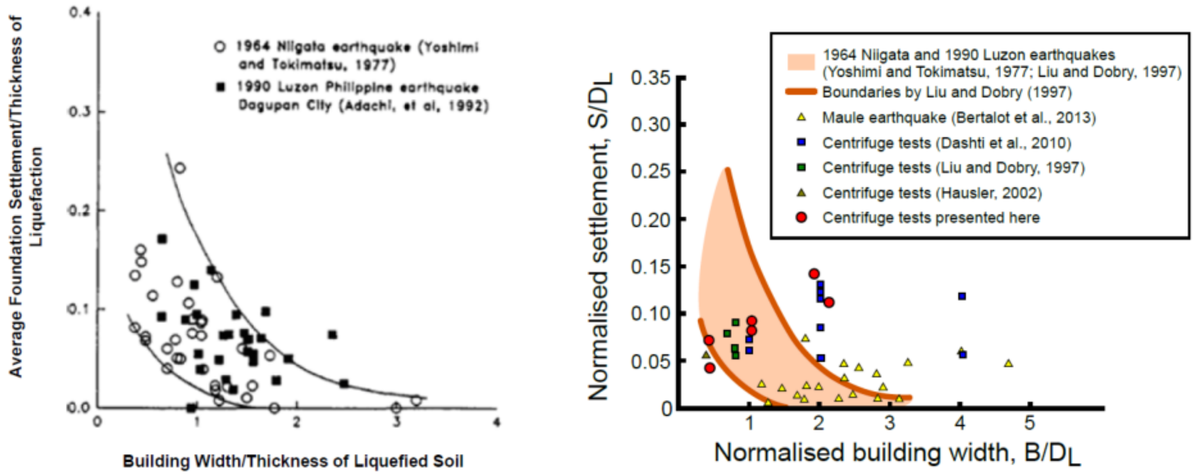

Figure 2-5: A). Normalised foundation settlement versus normalised foundation width based on available case histories (from Liu & Dobry 1997). B) The same chart (i.e. S= average foundation settlement, B=building width, D_L = thickness of liquefied soil) comparing case histories with published centrifuge tests (from Adamidis & Madabhushi 2017).

2.2.3 Relationship to Bearing Capacity

There is a relationship between the bearing capacity Factor of Safety (FS) and settlement of foundations post liquefaction (Naesgaard et al. 1998; Seed et al. 2003; Bray & Macedo, 2017):

- Foundations with FS < 1.0 are likely to exhibit punching shear failure and large settlements.
- Foundations of light to medium size structures that maintain a FS > 1.5 under earthquake loading exhibit negligible shear-induced settlement (D_s) (Bray & Macedo, 2017).
- A transition between the above two cases implies partial punching shear failure and associated settlements which can occur at isolated footings or on the edges of mat foundations and is complicated by dynamic interaction of structure and soil response (Seed et al. 2003). This suggests foundations with FS < 1.5 require further consideration.

Thus, as recommended by Bray & Macedo (2017), the assessment of the two-layer bearing problem (non-liquefied crust overlying liquefied soil) provides a useful screening tool to assess whether the shear-induced mechanism and resulting shear-induced settlements, D_s are likely to be significant or not. Methods to assess the two-layer problem range from simplified analytical equations for strip and square footings (Meyerhof & Hanna 1978, and other variants), as well as numerical analysis. A useful comparison of methods is presented by Stocks & Palmer (2019).

3 NEW SETTLEMENT PREDICTION MODELS

Two new settlement prediction models fill a knowledge & capability gap as identified by Seed et al. (2003). There are some similarities in approach, as both have been developed based on parametric studies using NLA, but also some key differences and assumptions between the two models that are discussed further below, in addition to discussion on the ground motion parameter *CAV* used as input to both models.

3.1 Bray & Macedo Method (BM17):

The simplified procedure assumes the three components of settlement (D_v, D_s and D_e) to be independent and evaluated separately, before summing to determine total settlement D_t. The method focuses on a new expression to estimate the D_s component, relying on existing simplified procedures for D_v - specifically the

Zhang et al. (2002) procedure with inherent limitations as discussed earlier, while acknowledging that as-yet no procedures exist to estimate D_e - they currently recommend correlation from liquefaction severity indices, and Hutabarat & Bray (2021) have recently proposed a new index with the aim of providing a basis for improved estimations (EPI -'ejecta potential index').

The D_s prediction equation was derived from statistical regression analysis on the results of 1,308 analyses conducted with the commercial numerical code FLAC v7, with PM4Sand as the constitutive model. Parametric variations considered included soil profile characteristics: Crust & liquefied layer thickness, soil density of liquefiable layer; as well as the shallow founded structures width, height and contact pressure also varied. The structures were all conventional frames on mat foundations, with periods ranging between 0.17 and 0.6s, corresponding to heights between 2 and 8 storeys. In total 105 different models were considered, with 12 ground motion records used as input for the majority of analyses, with a further 24 considered for select models, including amplitude scaling to consider high intensity shaking.

The equation for the D_s component requires a number of inputs, including the applied bearing pressure, q; footing width B; thickness of liquefaction H_L; a new liquefaction building settlement index; and two earthquake ground motion parameters: CAV_{DP} – a minimum threshold variant of Cumulative Absolute Velocity (a parameter discussed further subsequently), and S_{a1} the spectral acceleration at 1s period. Parameters *LBS* and H_L are both derived from the results of the simplified liquefaction triggering method, and commercial software *CLiq* allows for the calculation of BM17's D_s parameter for a given CPT, provided the user also input B, q, CAV_{DP} and S_{a1}.

BM17 present application to centrifuge data (finding predictions fall within 0.5 – 2x measured settlements), to 19 case histories including many from Christchurch, and further note favourable comparison to 3D numerical analyses by Karimi & Dashti (2013).

3.2 Bullock et al. (2019) Method (BEA19)

BEA19 present prediction equations for both building settlement and tilt, based on statistical regression analyses on the results of a large parametric study using 3D NLA using the open source code OpenSees with the PDMY02 constitutive model undertaken by Karimi et al. (2018). A total of 421 models were considered each subjected to 150 ground motion records for a total of 63,150 analyses. Some 30,278 resulted in building settlements in excess of 10mm and subsequently used in the regression. Geometric variations of the structure (mass, stiffness, height, bearing pressure, foundation geometry), and soil profile (site period, thickness of crust, soil density, layering -including density and permeability variations and hence extent of triggering) were considered within the study. The resulting settlement predictive equation includes a number of terms that capture the effects of soil profile characteristics, foundation and structure:

- The soil profile must be firstly assessed for susceptibility of individual layers to liquefaction triggering using plasticity-based compositional criteria (refer MBIE/NZGS 2016 guidelines), and for susceptible layers the normalised SPT $N_{1,60}$ or CPT q_{c1N} data for clean sands determined for each layer. A triggering assessment is *not* undertaken/required for input and therefore whether soils partially soften or fully liquefy, and the extent that this is affected by building response is inherently captured by the regression model. Density and depth weighting factors for settlement contribution of the various layers are incorporated within the equations.
- The ground motion parameter Cumulative Absolute Velocity, *CAV* on rock conditions (i.e. no site amplification factoring) is required as input, and no site response analysis is undertaken, or site class specified. No further ground motion inputs are required.
- The effect of foundation width, length, bearing pressure, depth is considered, and incorporates weighting factors if liquefiable layers are present or not within the foundation's depth of influence.
- The building structure characteristics of height of structure and mass are considered to reflect building inertial response.

The final reported R^2 value was 0.86. The model was subsequently corrected based on performance compared to some 50 selected case histories of mat foundation structures, the majority from the 1999 Kocaeli & Düzce (Turkey), and 2010 Chile earthquakes. Comparison was made to all case histories including non-mat shallow foundations (87 cases total) and showed little bias with magnitude of settlement, though more scatter with the latter. The many inputs to the equation make it more complex to implement, and the BEA19 authors provide an Excel spreadsheet they developed to improve take up of the model. It includes a Ground Motion Prediction Equation (GMPE) for *CAV* on rock, provided the earthquake source inputs are known (magnitude, distance, fault sense, etc).

3.3 The CAV ground motion parameter

For the BM17 and BEA19 settlement prediction models, both methods require a new ground motion input parameter, Cumulative Absolute Velocity (*CAV*). *CAV* is defined as the integration of the absolute value of acceleration over the duration of the earthquake and has units of m/s (but is not related to velocity). *CAV* was developed in the US by the Electric Power Research Institute (EPRI) as a damage parameter to be used to determine when to initiate a safe shutdown of Nuclear Power Plants. It has been shown to correlate well with observations of earthquake damage and is more predictable a variable than alternatives such as Arias Intensity (Campbell & Bozorgnia 2012). Variants of *CAV* that feature a threshold acceleration have been shown to correlate well to excess pore water pressure generation in NLA of liquefaction problems (Kramer & Mitchell 2006, Montgomery & Boulanger 2016). Independently both BM17, and BEA19 also found that a *CAV* based parameter offered a better candidate intensity measure to predict liquefaction-induced settlements of buildings. BM17 adopted CAV_{DP} (a threshold variant), while BEA19 adopted *CAV* on rock.

Note that the BM17 equation for D_s also requires S_{a1}, and PGA and Magnitude, as it relies on a conventional liquefaction hazard assessment for some inputs. The use of multiple ground motion input parameters complicates matters, as different distributions of events determine the hazard for each parameter. Adopting uniform hazard estimates (e.g. from code-derived spectra values or Probabilistic Seismic Hazard Analysis (PSHA)) would be expected to result in conservative estimates of settlement.

4 RECOMMENDATIONS FOR APPLICATION OF SIMPLIFIED APPROACHES

4.1 Establish Suitability of Approach

Prior to the adoption of the simplified methods discussed here we recommend a specific applicability review is undertaken addressing firstly whether, in design situations allowing the building to be subject to liquefaction induced settlement (for a particular limit state) is consistent with the design brief and structural form; whether the proposed delivery mechanism (e.g. traditional, ECI, design & build) consistent with the achieving the Principal's requirements in terms of performance at a particular limit state; if there are any statutory obligations that may preclude founding a building on potentially liquefiable soils; and whether or not the site is potentially subject to lateral ground movements, noting that none of the methods discussed within this paper are applicable to either the assessment of settlement associated with lateral spreading, or the lateral movement itself.

4.2 Shear Settlement estimates (D_s)

At an FS bearing <1.5, ratcheting is expected to start to become significant. The authors suggest the use of FS bearing >1.5 as a pragmatic screening threshold for the applicability of the simplified methods to new buildings. If design of new structures with FS bearing lower than this level is proposed (e.g. a 'performance based' design for sensitive, high importance or high value structures) consideration should be given to the use of the new simplified building settlement prediction models to inform design and/or undertaking NLA to capture building and site specific response characteristics.

4.3 Free field (D_v) and ejecta (D_e) settlement estimates

Free field settlement estimates are very easy to calculate using semi-empirical CPT-based methods, but care is required to interrogate the liquefaction assessment that it is based upon, and to systematically screen out layers that are unlikely to trigger and not contribute significantly to volumetric settlements, D_v. The following present general recommendations to aid but not supplant engineering judgement:

1. Calibrate using site-specific data the measured *FC* to CPT I_c relationship (via C_{FC} parameter), and I_c cut-off (via Atterberg Limit test data) adopted in the liquefaction triggering assessment, as recommended in the liquefaction assessment guidelines (MBIE/ NZGS 2016). This requires close placement of a representative CPT and borehole, and in so doing may identify a particular soil that varies from the default relationship, or possibly a drift in the friction sleeve measurement that affects the I_c calculation and hence *FC* prediction, and consequently impact the liquefaction assessment and resulting settlement predictions.

2. Review and potentially eliminate highly inter-bedded soil layers where thin layers of sand alternate with non-liquefiable silt-clay layers (refer Boulanger et al. 2016).

    - Highly interbedded layers may inhibit the transfer of settlement to the ground surface due to the high spatial heterogeneity of such deposits, which promote soil arching. Further, such deposits tend to be formed in low energy environments and contain fines, noting that settlements calculated from soil layers high in fines content are very likely over-predicted by the simplified methods.

3. Consider only soil layers within the upper 10m profile.

    - While some researchers have suggested a depth weighting factor to counter the observed bias in predictions (Cetin et al. 2009; Geyin & Maurer 2019), it is not yet conclusive on the appropriate function to apply, and while the bias may be reduced, the scatter between observations and predictions increases (Geyin & Maurer 2019). Note that the BEA19 prediction model may be utilised without any such limitation as it incorporates depth weighting inherently in the functional form, and may with suitable light-foundation loads, be used as a check on simplified free field estimates of settlement.

    - Consider the likelihood of ejecta being significant – a function of severity of liquefaction and non-liquefied crust thickness, using indices such as LSN or LPI_{ISH} (van Ballegooy et al. 2014; Maurer et al. 2015). As liquefaction ejecta increases, Geyin & Maurer (2019) showed the depth-bias was reduced, implying that both components (D_v and D_e) need to be considered to establish free field settlement predictions more accurately, for example:

        - for thin non-liquefied crusts (where the ejecta potential is higher, or where buildings may puncture the crust and facilitate ejecta), and where larger magnitude D_v is predicted (i.e. > 60mm), D_v may be considered to incorporate D_e, precluding the need for depth-weighting or a separate evaluation of D_e related settlement.

        - for thicker non-liquefied crusts (where ejecta potential is lower), larger settlement predictions may still require a depth-weighting correction, but further research work is required to establish a suitable function that should be applied, and to advise on appropriate magnitudes of ejecta settlement. Judgement must be applied in the interim, and again the BEA19 prediction model may provide a useful tool to calibrate volumetric & ejecta related settlement predictions for either of the above noted cases.

4.4 For assessing differential settlements and residual tilt.

1. Consider from a number of CPT across the site of interest the range and average (or with sufficient data, the median) of total settlement predictions. This, along with a suitable allowance for uncertainty, will provide an indication of variability in settlement that may be expressed at the ground surface, affecting footings and services. A rule of thumb sometimes used in design or assessments estimating differential settlement between structural support to be ~ ½ total settlement (e.g. Anderson et al. 2007) should be considered only as a preliminary estimate as it does not consider many of the factors that influence surface expression of settlement (discussed subsequently), and is necessarily limited by available ground investigation data.

2. Consider the thickness of the soil crust. Greater thickness will inhibit the transfer of settlement to the surface, both in terms of total settlements but perhaps more significantly the distance over which differential settlements should be considered to affect structures and buried infrastructure (refer Montgomery & Boulanger, 2016). Numerical analysis will likely provide better insights than the aforementioned rules of thumb to evaluate the expression of differential settlement.

3. Consider both the results of BM17 and BEA19 prediction models for a range of relevant CPT as inputs across a site (as for (1) listed above), to obtain the range of total building settlements expected at the given design earthquake considered. NB: The BEA19 prediction model also provides tilt estimates, which may be of particular interest for tall, narrow structures. In our experience BM17 tends to provide higher estimates than BEA19, likely on account of differences in how D_v in particular is evaluated, as both adopt a similar basis for the D_s component. NB: The correction factor BEA19 apply to address bias between predicted settlements and case histories (which incorporate both D_v and D_e components) to some degree accounts for the inability of numerical analyses to accurately capture these aspects.

4. Assessment of the consequences of differential settlement and damage criteria should generally follow industry standard guidance, as for other sources of differential settlement, e.g. Burland (1997), ICE (2012), Rankine (1988), CIRIA C796 (Schoor et al. 2021).

4.5 Ground motion input parameters for BM17 & BEA19

Probabilistic Seismic Hazard Assessment (PSHA) derived values of CAV_{DP} and CAV on rock, are not provided in the New Zealand loadings standards or associated documents currently, which is an impediment to application of these methods for engineering design and assessments. A suitable means to derive these parameters requires a site-specific hazard assessment currently, to either calculate CAV from a PSHA, or to use conditional estimates based on the hazard at other spectral ordinates, such as PGA, utilising a conditional intensity measure approach (e.g. Baker 2011, Bradley 2010). Taylor et al. (2021) provide conditional hazard estimates of CAV and CAV_{DP} values for selected sites in New Zealand that vary with return period and site class (paper currently under review for publication in the Bulletin of the NZSEE).

An alternative approach would be to consider scenario earthquake events from select dominant fault sources in the region, where these are published, with the required characteristics input to published Ground Motion Prediction Equations for CAV or CAV_{DP}, as well as PGA and S_{a1}. Adopting median + 1 standard deviation is typical for deterministic assessments. This is less rigorous than based on PSHA hazard estimates which is the preferred approach for design in accordance with NZS1170.5 (apart from low seismicity regions) but may be sufficient for some projects in the absence of a site-specific study that incorporates CAV hazard predictions.

5 SUMMARY & CONCLUSIONS

This paper presents a review of procedures to evaluate the liquefaction-induced building settlement for shallow founded structures. It summarises recent developments in the last decade, including new procedures to consider the combined mechanisms that contribute to building settlement – volumetric, shear-induced and ejecta related settlements. The new models fill a much-needed gap in the tools available to practitioners. Significant uncertainties and limitations with the approaches remain, and these have been discussed and suggested guidance given. Both new models are recommended for application now, and by considering both the epistemic uncertainty associated with any one prediction model may be considered to some degree. Further work is required to improve the assessment of settlement in soils containing fines, including soils with fine interbeds, and in the quantifying of settlements due to sand ejecta, and further research efforts in these areas are to be encouraged.

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