# Performance assessment of shallow founded buildings on liquefiable soils

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## ABSTRACT

In evaluating foundation options for sites featuring liquefiable soils, shallow raft foundations may remain a suitable and preferred solution, provided the seismic performance can be demonstrated to be adequate. In evaluating the feasibility of this solution, it becomes critical to assess the ground and foundation performance to ensure it will reasonably meet code requirements and client performance objectives without overengineering. This is hampered by the large uncertainties in making settlement predictions for future earthquakes using the available simplified empirical and analytical methods, as well as the crudeness of geotechnical investigation methods.

This paper presents a design case study for a site in Christchurch where a range of settlement predictions were obtained in a sensitivity study using simplified methods, leveraging the advantage that predictions may be compared to observations of the 2010-2011 Canterbury Earthquake Sequence. Insights into the limitations in practical application of published methods of assessment are discussed with the aim of gauging where the methods may over- or underestimate performance to inform the selection of a 'best estimate' for design, and from there, determining a suitable range of uncertainty to consider. Findings show the highest variation is due to assigned depth of evaluation and estimation of ejecta-induced settlement.

## 1 INTRODUCTION

The impact of significant seismic shaking during the 2010-2011 Canterbury Earthquake Sequence on ground performance and shallow founded structures affected by soil liquefaction has been well documented, both by observations immediately following the events (Cubrinovski et al. 2011a; Cubrinovski et al. 2011b; Bray et al. 2014) as well as subsequent research efforts to evaluate building performance involving advanced dynamic numerical analysis of case histories of specific structures (Bray & Macedo 2017; Luque & Bray 2020). Adverse performance was observed for a number of structures where soil liquefaction had occurred in shallow soil layers within influence of the structure foundation, typically resulting in total and differential settlements, and resulting distress to the structure itself.

Seismically induced settlement of buildings on level ground is understood to be the result of complex mechanisms that occur between soil and structure both during the event as well as post-shaking, and these interactions contribute to the final settlement of the structure. Bray & Dashti (2014), based on case history observations from earthquakes and centrifuge experiments, identified a number of interactive mechanisms (refer Figure 1). In summary, three components were considered to contribute to seismic-induced building settlements (Taylor et al. 2021):

- **Volumetric-induced settlement (also known as 'free-field' settlement), $D_v$:** free-field settlement including sedimentation and re-consolidation of liquefied soils post-shaking as well as 'shake down' settlement of dry granular soils and poorly compacted fills. $D_v$ is evaluated by commonly applied methods such as Zhang et al. (2002), Idriss & Boulanger (2008), and Tokimatsu & Seed (1987).
- **Shear-induced settlements, $D_s$:** further settlement resulting from the additional shear strain induced in founding soils by the building, often by rocking, leading to punching shear type bearing failures or soil-structure-interaction (SSI) ratcheting. $D_s$ is quantified through analytical methods such as Bray and Macedo (2017) and Bullock et al. (2019), or through dynamic numerical analysis (e.g., Karimi & Dashti 2016a, 2016b; Luque & Bray 2020).
- **Ejecta-induced settlements, $D_e$:** Settlement occurring from loss of soil below the foundations due to liquefied material escaping to the surface, driven by high excess pore water pressures induced by oscillating imposed building loads. $D_e$ is estimated through ground failure indices and experience. Improved assessment procedures are a topic of current research, with Hutabarat & Bray (2022) recently proposing a qualitative liquefaction ejecta severity assessment method for level sites.

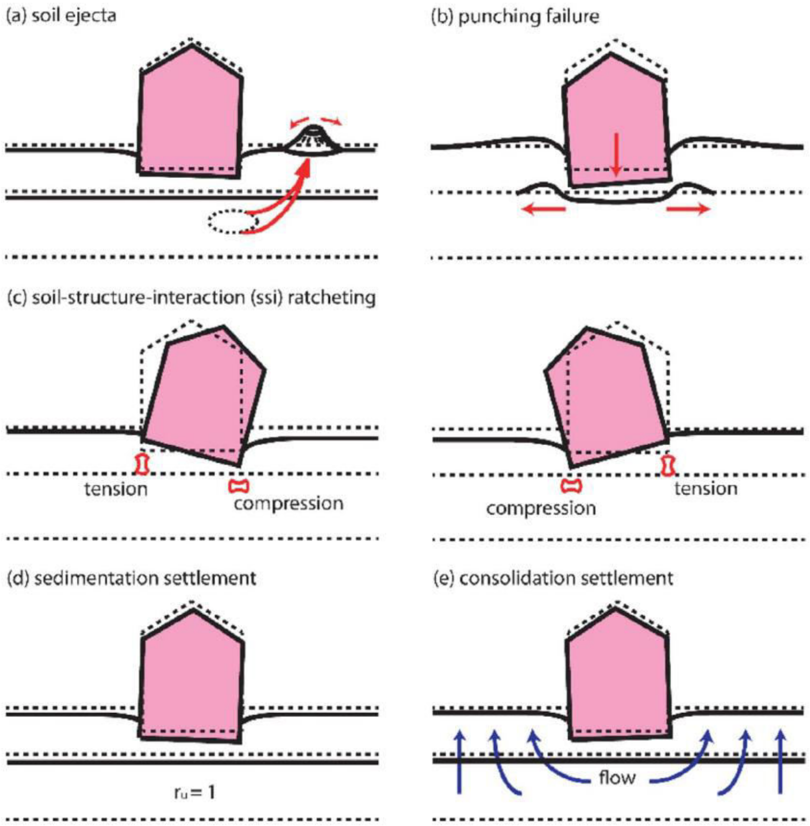


*Figure 1: Liquefaction-induced building displacement mechanisms: a) ground loss due to soil ejecta; shear induced settlement from (b) punching failure, or (c) soil-structure-interaction (SSI) ratcheting; and volumetric-induced settlement from (d) sedimentation or (e) post-liquefaction reconsolidation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

Geotechnical engineering practitioners are required to consider the performance of the ground at a site and its suitability for founding proposed structures on firstly shallow foundations which are the most economical solution, or an alternative solution if required, such as piled foundations or ground improvement which add significant cost and programme to the delivery of a project (e.g., Ministry of Education 2020). As part of the assessment of the suitability of shallow founded structures, consideration as to the performance in earthquakes is required, and guidance is provided in NZGS/MBIE (2021) Earthquake Geotechnical Engineering Module 4.

Key design considerations for the performance-based assessment include determining what level of shaking intensity will result in a 'step-change' in performance of the shallow founded structures (i.e., large differential settlements), the magnitude of the step-change if applicable plus associated consequences to the structure performance, and whether or not the performance meets both building code requirements and the project-specific performance objectives of the client (refer NZGS/MBIE 2021 Module 1 for brief discussion on performance-based design and the limitations of the limit-state design framework in AS/NZS 1170.0:2002).

This paper presents a case study discussing the evaluation of shallow foundation performance for proposed new buildings in a light commercial / industrial area south of the Christchurch central business district (CBD). The traditional approach to assessing the impact of liquefaction-induced settlement on shallow foundations has involved estimating free-field ground settlements alone, and in some cases to consider additional settlements solely due to the additional weight of the building on liquefied soils with temporarily reduced stiffness and strength to assess possible shear-induced settlements. These have been shown not to accurately account for all building-induced settlement mechanisms such as SSI ratcheting or ejecta loss, or the complex interaction between excess pore pressure fluctuation and building response.

Recently new models have been developed that attempt to incorporate these effects and provide better estimations of building performance, such as Bray & Macedo (2017) and Bullock, et. al. (2019). This case study serves to compare the available methods of settlement estimation as part of a sensitivity study, including discussion on practical application and considerations of over- and under-prediction in practice. The benefits of undertaking comparisons at sites in Christchurch is the available performance of the site under recent earthquakes which offers invaluable feedback in scrutinising the resulting estimates.

## 2 SITE AND SEISMIC CHARACTERISATION

### 2.1 Geology and ground model

The relevant published geological map of the Christchurch Urban Area (Brown & Weeber, 1992) shows the site of interest to be immediately underlain by Holocene (<10,000 years old) sediments of the Springston and Christchurch Formations in succession. The subsoil profile developed for the geotechnical assessment was informed by site-specific ground investigations (boreholes with SPT) and historic data compiled from nearby locations. Laboratory testing of the silts confirm low plasticity.

### 2.2 Proposed development

The proposed development comprises a braced steel Main Building, and a lightweight steel Store Building. The subsoil profile is susceptible to liquefaction for the full 20m of investigation depth. Through discussions with the structural team, two initial foundations options were considered: bored piles or a ground improvement solution in the form of a cement-soil lattice to form intersecting in-ground walls. It became quickly evident that the cost of these options was prohibitive for what are modest scale structures, so further refinement of the estimated site and building settlement performance would be necessary to comfortably adopt a shallow foundation solution. The proposed foundation solution is a reinforced structural raft for the

Main Building, 600mm deep, and a system of tied ground beams for the Store Building, 800mm deep. The concrete foundations are to be underlain by a nominal 500mm of gravel to mitigate effects from liquefaction reaching the surface.

## 2.3 Site performance in the 2010-2011 CES

During the 2010-2011 Canterbury Earthquake Sequence (CES), Christchurch experienced four significant earthquakes and thousands of aftershocks with widespread damage across the city. Performance of the site during these events was reviewed through data provided on the New Zealand Geotechnical Database (NZGD 2022). Aerial imagery immediately following the earthquakes showed liquefaction ejecta was visible in a few discrete locations of the site during the 4-Sept-2010 Darfield event and moderate quantities of ejecta observed following the 22-Feb-2011 Christchurch event over the northern and eastern half of the site.

Vertical ground surface changes measured using LiDAR for three of the CES earthquakes are shown in Figure 2 along with footprints of the buildings existing at the time and the two buildings proposed to be constructed. Note, the pre-CES LiDAR data was not as high quality when comparing to the first event, and some anomalies are evident. These maps present a rough estimate order of magnitude of ground surface elevation change and have been used to inform an assessment of general over- or under-prediction of the methods implemented in this study. This real-life performance is considered alongside ejecta severity observations in each event and the calculated ground damage performance indicators from applied seismic loading scenarios.

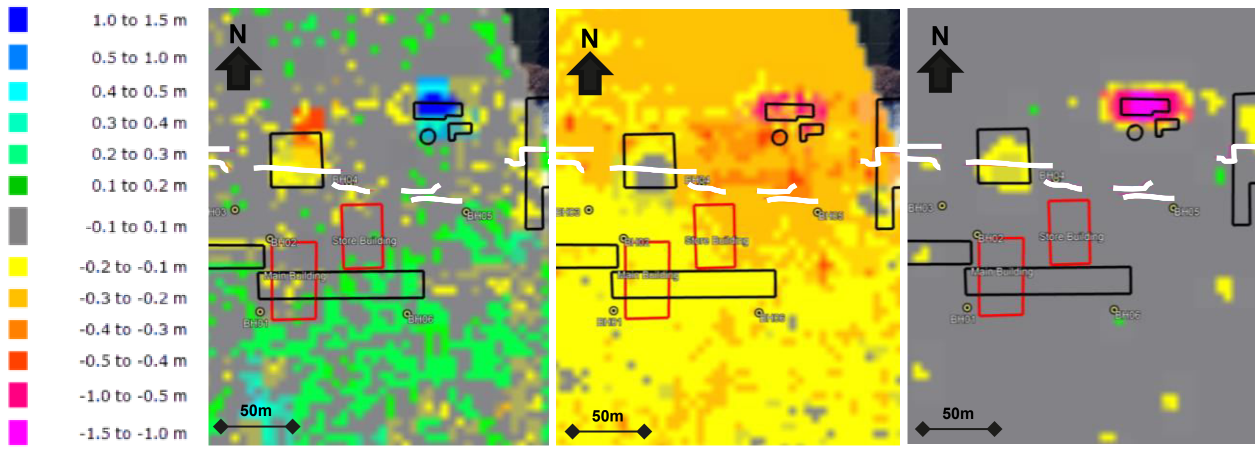


*Figure 2: Vertical elevation changes due to events on (left to right) 4-Sep-2010, 22-Feb-2011, 13-Jun-2011. Black lines: Existing building footprints. Red lines: Proposed new building footprints. White lines: mapped ground cracking (NZGD, 2022).*

## 2.4 Design ground motions

### 2.4.1 Cumulative Absolute Velocity (*CAV*)

Two of the new settlement assessment methods considered in this assessment require ground motion hazard inputs in the form of Cumulative Absolute Velocity (*CAV*). *CAV* is defined as the integral of the absolute value of acceleration over the duration of the earthquake, and thus incorporates both the shaking amplitude and its duration within a single parameter. It has been shown to correlate well to structural damage as well as the development of excess pore water pressure and liquefaction triggering (Campbell & Bozorgnia 2019; Taylor et al. 2021).

Following the CES, updated ground motions for use in engineering design were specified by the Canterbury Earthquake Recovery Authority (CERA) and subsequently MBIE for the rebuild of Christchurch. The method adopted to determine a *CAV* that is compatible with the MBIE design values was to:

1. Review Peak Ground Acceleration (*PGA)* seismic hazard disaggregation data to estimate *CAV* for the corresponding mean event (*PGA,* and mean magnitude, *M* from NZGS/MBIE guidelines and mean rupture distance, $R_{rup}$ from disaggregation data presented by Bradley (2014).
2. For the MBIE-provided *PGA* and *M* values and assessed weighted mean $R_{rup}$, determine epsilon ε using the Bradley (2013) empirical ground motion model (GMM), adopting a reverse fault mechanism consistent with the predominant seismotectonic regime. Published correlation coefficients between the residuals of *PGA* and *CAV* were used to adjust the ε (*PGA*) values to appropriate values for the calculation of *CAV* (Bradley 2012).
3. Using four recently developed GMMs for *CAV*, each with 25% weighting, calculate a weighted average *CAV* value corresponding to each *M,* $R_{rup}$, ε combination, for different soil conditions (i.e., adopting VS30 values corresponding to the applicable NEHRP site class). The *CAV* GMMs were: Campbell & Bozorgnia (2019); Bullock (2019); Bullock et al. (2021), and a conditional GMM for *CAV* developed by Mao (2020) with inputs provided by the Bradley (2013) GMM; providing a mix of international 'NGA2West'-based and New Zealand-specific models.

This approach adopts necessary simplifications due to limited information available for the specified hazard values but is considered reasonable within the context of the assessments being undertaken.

### 2.4.2 Spectral acceleration at 1s period, compatible with MBIE *PGA* hazard values

One of the building settlement prediction models (Bray & Macedo 2017) requires spectral acceleration at a vibration period of 1s [*Sa(1s)*] as a concurrent ground motion input along with *PGA* (for liquefaction assessment) and *CAV*. It is noted that these *Sa(1s)* values will differ from those provided by NZS1170.5 which are based on uniform hazard, and therefore come from a different distribution of source events than *PGA*. The *Sa(1s)* values compatible with MBIE prescribed *PGA* hazards were calculated in a similar manner to the *CAV* values above and is essentially the same approach as the Conditional Mean Spectrum used to develop design spectra for ground motion selection (Baker 2011). The Bradley (2013) GMM was used to calculate *Sa(1s)*, with the correlation coefficient adopted between *PGA* and *Sa(1s)* from Bradley (2011).

### 2.4.3 Seismic design parameters

The recent update to the NZGS/MBIE (2021) Module 1 guideline provides two earthquake scenarios for SLS and one for ULS considering Importance Level 2 (IL2) structures. The SLS1a, SLS1b, and ULS earthquake scenarios correspond roughly to the events on 4-Sept-2010 (*PGA* 0.22g, Mw 7.1); 13-Jun-2011 (*PGA* 0.235g, Mw 6.0); and 22-Feb-2011 (*PGA* 0.455g, Mw 6.2), respectively (refer Table 1 for seismic loading).

*Table 1: Ground motion parameters for the liquefaction induced settlement assessment*

| **Design Event** | **APE* (1/ years)** | **PGA (g)** | **Magnitude (M)** | **Sa(1s)** (g)** | ***CAV* (g-s) by Subsoil Class:** | |
|---|---|---|---|---|---|---|
| | | | | | **Class A/B** | **Class D** |
| SLS1a | 1/25 | 0.13 | 7.5 | 0.13 | 0.50 | 0.95 |
| SLS1b | 1/25 | 0.19 | 6.0 | 0.08 | 0.25 | 0.50 |
| ULS | 1/500 | 0.35 | 7.5 | 0.48 | 1.27 | 2.35 |

* Annual Probability of Exceedance ** Spectral Acceleration (T=1s) conditional on PGA, for subsoil Class D

# 3 FREE-FIELD LIQUEFACTION ASSESSMENT

## 3.1 Liquefaction triggering

NZGS/MBIE (2021) Earthquake Geotechnical Engineering Module guidance was followed, adopting the Boulanger & Idriss (2014) liquefaction triggering assessment method to assess the hazard using available site-specific geotechnical data collated from the site; being six boreholes with Standard Penetrometer Testing (SPT). The assessment indicates site-wide liquefaction is likely to occur under both SLS earthquake loading cases. The analysis showed liquefaction triggering for SLS-level of shaking at depths ranging from 6.5m to 10m below ground level (bgl). Under ULS earthquake loading, liquefaction triggering is expected to occur within 2m of the ground surface.

## 3.2 Free-field settlements

Conventionally, the assessment of liquefaction-induced free-field (FF) vertical settlements adopt procedures described by Tokimatsu & Seed (1987), Ishihara & Yoshimine (1992), Zhang et al. (2002), or Idriss & Boulanger (2008). For this study the Zhang et al. (2002) procedure has been adopted.

Recently Geyin & Maurer (2019) compiled and analysed 1,013 case-histories from the 2011 Christchurch earthquakes to quantify FF settlement. They compared the predicted settlements using common methods including Zhang et al. against observations of ground settlements from LiDAR survey data and found that FF settlements were typically overpredicted when they exceeded 60mm, attributed in part to assuming all layers in the profile contributed equally to the resulting settlement at the ground surface. Cetin et al. (2009) identifies a number of mechanisms why this is likely to be an overly conservative assumption.

Geyin & Maurer (2019) applied a simple depth-weighting (DW) factor to reduce the influence of deeper layers on the expression of settlement at the ground surface but the large uncertainty in estimated magnitude of FF settlements remained, some of which may be attributed to other processes such as liquefaction ejecta and its removal during the clean-up soon after the earthquakes. The depth-weighting concept will be investigated further in this study.

# 4 PREDICTING SURFACE MANIFESTATION AND GROUND DAMAGE POTENTIAL

## 4.1 Seismic performance screening evaluation

Ishihara (1985) developed empirical curves, based on observations from earthquakes featuring liquefaction in Japan to allow for the prediction of the minimum thickness of the non-liquefied crust (above the liquefiable soils) required to suppress liquefaction manifestation at the ground surface (i.e., ground cracking, sediment ejecta). Using these curves, liquefaction-induced ground damage is not expected under SLS loading but is expected under ULS loading. These predictions inform the expectations of liquefaction surface manifestation in a binary fashion, subsequent studies have proposed more quantitative methods.

An evaluation of bearing failure using a two-layer model (crust over liquefied layer) can act as a screening tool to indicate whether shear-induced settlements are likely to be significant or not, where factor of safety, $FS$, is calculated as the ultimate bearing capacity over applied load. Bray & Macedo (2017) found that foundations with an assessed $FS < 1.0$ are likely to exhibit punching shear failure and large settlements, whereas foundations that maintain a $FS > 1.5$ under earthquake loading exhibit negligible shear-induced settlement. A transition between $FS < 1.0$ and $FS > 1.5$ implies partial punching shear failure complicated by dynamic interaction of structure and soil response, therefore $FS = 1.5$ is adopted as a cut-off value, below which the shear-induced settlements should be further evaluated. At this site, both of the proposed buildings were assessed to have $FS$ greater than 1.5 for SLS loading, and $FS$ less than 1.5 for ULS loading.

## 4.2 Performance indicators and historic performance

Guidance in MBIE/NZGS (2021) Module 3 outlines a total seismic settlement performance correlated to ranges of two empirical indices of ground performance: Liquefaction Potential Index (*LPI*) developed by Iwasaki et al. (1982), and the Liquefaction Severity Number (*LSN*) introduced by van Ballegooy et al. (2014) which incorporates depth-weighting. Both indices may be calculated from conventional procedures to assess liquefaction triggering of the soil profile and estimate the resulting FF settlements. In addition to total settlement, however, we want to be able to estimate ejecta-induced settlement through empirical methods, for input into the Bray & Macedo (2017) method. The current best practice is to retroactively quantify the volume of ejecta observed on the surface after a seismic event.

By inspecting four case histories across Christchurch (Luque & Bray, 2017; Luque & Bray, 2020) we can associate estimated ejecta-induced settlement post-event to vertical LiDAR data and observed liquefaction severity, see Table 2. This can be extrapolated to correspond with performance predicted by *LPI* and *LSN*.

*Table 2: Observations of liquefaction manifestation during CES events for documented case studies of building sites in the Christchurch CBD (refer main text), and the project site (this study). Source data: Aerials and LiDAR settlements (NZGD, 2022).*

| **Site** | **4 Sep 2010 [Similar to SLS1a]** | | | **13 Jun 2011 [Similar to SLS1b]** | | | **22 Feb 2011 [Similar to ULS]** | | |
|---|---|---|---|---|---|---|---|---|---|
| | **Ejecta Severity*** | **FF $D_v$ (mm)**** | **$D_e$ est. (mm)***** | **Ejecta Severity*** | **FF $D_v$ (mm)**** | **$D_e$ est. (mm)***** | **Ejecta Severity*** | **FF $D_v$ (mm)**** | **$D_e$ est. (mm)***** |
| FTG-7 | None - Moderate | 0 - 400 | 0 | Moderate - Severe | 0 - 200 | 50 - 100 | Moderate - Significant | 300 - 1000 | 40 - 80 |
| CTH | Minor | 0 - 200 | 0 | Moderate - Severe | 0 - 200 | 0 | Moderate - Severe | 100 - 1000 | 0 - 100 |
| CTUC | None | 0 - 200 | N/A | Moderate - Severe | 100 - 1000 | N/A | Minor - Moderate | 100 - 500 | 70 - 150 |
| PWC | None | 0 | N/A | Moderate - Severe | 0 | N/A | None - Minor | 100 - 400 | N/A |
| Study Site | None | 0 - 200 | - | Minor - Severe | 0 - 200 | - | Minor - Severe | 100 - 400 | - |
| Expected Study Site Settlements | Total: 0 - 200 mm<br>Ejecta-induced: Negligible | | | Total: 0 - 200 mm<br>Ejecta-induced: 0 - 50 | | | Total: 200+ mm<br>Ejecta-induced: 0 - 100 | | |

* Qualitative ejecta severity from observations (aerial imagery)

** Free field settlement ($D_v$) range from processed LiDAR data (pre- and post-event, tectonic movements removed)

*** Estimates from published case studies of building sites post CES

Table 3 presents *LPI* and *LSN* correlated to estimated total settlement from Module 3 and estimated ejecta-induced settlement based on case history estimation and NZGD observations, proposing a set of criteria on which to assign ejecta-induced settlement estimations from calculated *LPI* and *LSN*. Finally, Table 4 applies these criteria to site-specific *LPI* and *LSN* calculated for the project site to estimate a range of total settlements and ejecta-induced settlements that might be expected for each limit state earthquake scenario.

The predictions in Table 4 are consistent with observations from the CES as well as the prediction by Ishihara (1985) of surface manifestations to be expected under ULS conditions but not SLS conditions.

Note the variation in estimated ejecta-induced settlement from case studies is not perfectly consistent with observations of liquefaction severity or vertical LiDAR data. As such, there must be other factors or processes in the development of liquefaction, and the dissipation of excess pore water pressures that affect the manifestation of ejecta but are not fully captured by simple damage index parameters like *LPI* or *LSN* (e.g., Hutabarat & Bray 2022), and the values adopted here are representative averages as a result of a qualitative comparison. This source of inaccuracy in estimated ejecta-induced settlement is considered when determining the 'best estimate' total seismic settlement for the site.

*Table 3: Settlement Performance Indicator Comparison*

| Liquefaction Effects | *LPI* | *LSN* | Total Settlement (mm) | Ejecta-Induced Settlement (mm) |
|---|---|---|---|---|
| Minor - Moderate | 0 - 15 | 0 - 15 | Negligible - Small | 0 |
| High | 5 - 15 | 15 - 30 | 100 - 200 | 50 |
| Severe | 15 - 25 | 30 - 60 | 200+ | 100 |
| Severe | 25+ | 60+ | Significant | 150 |

*Table 4: Estimated Site Performance from LPI and LSN*

| Ground damage index / Est. settlement (mm) | SLS1a | SLS1b | ULS |
|---|---|---|---|
| *LPI* | <5 | 5 | 20 - 35 |
| *LSN* | 10 - 20 | 15 - 30 | 40 - 75 |
| Estimated total liquefaction-induced settlement | Small | 100 - 200 | 200+ |
| Estimated ejecta-induced settlement component | 0 - 50 | 50 | 100 - 150 |

# 5 SEMI-EMPIRICAL BUILDING SETTLEMENT PREDICTION MODELS

## 5.1 Bullock method for liquefaction-induced building settlement

The 'Bullock method' (Bullock et al., 2019) is based on the results of an extensive parametric study using 3D dynamic effective stress analyses using the non-commercial research finite element code OpenSEES. The results were curve fit using statistical methods to an empirical prediction equation. The equation was further calibrated to the results of centrifuge experiments as well as a database of case history observations including Christchurch to ensure the model predictions reflected 'real world physics.' The model calculates the total building settlement and tilt, i.e., all mechanisms from volumetric strains ($D_v$), deviatoric strains ($D_s$), and ejecta ($D_e$) are considered together in the one calculation, and it is implied that any depth weighting of $D_v$ contribution is implicitly considered.

Inputs for this method include influences from the soil profile (, the foundation (applied pressure), the structure, and the design earthquake (*CAV* on rock). The outputs are in the form of settlement prediction – median estimate plus variability as expressed by the model's standard deviation, hence 84$^{th}$ percentile estimates may be calculated (mean + 1σ) to capture the model uncertainty in the performance assessment.

### 5.2 Bray method for for liquefaction-induced building settlement

The 'Bray method' (Bray & Macedo 2017) is a simplified procedure based on 1,300 nonlinear 2D dynamic effective stress analyses undertaken in commercial program FLAC, field case histories, and centrifuge test results to estimate the shear-induced component ($D_s$) of liquefaction-induced building settlement. Seismic loading inputs to calculate shear-induced settlements ($D_s$) using the Bray method include *CAV* on soil and spectral acceleration, *Sa(1s)*, as discussed in previous sections. The calculation considers incremental shear strains and the *FS* against liquefaction throughout the depth of the evaluated soil column following methodology presented by Boulanger & Idriss (2014).

Total liquefaction-induced settlement, $D_t$ is a summation of separate calculations for each component ($D_s + D_v + D_e$). The volumetric-induced (free-field) settlement, $D_v$, is evaluated through semi-empirical procedures using CPT or SPT data from site. Ejecta-induced settlement, $D_e$, is estimated through observations from case histories, correlated to liquefaction damage indices as discussed previously. Although the method allows for uncertainty in $D_s$ to be captured via the model's standard deviation, the uncertainty in $D_t$ using this method appears dominated particularly by the $D_v$ component and uncertainty around the expression of FF settlements at the ground surface. In this study, the cut-off depth for the evaluation of $D_v$ has been set at 10m and 20m for comparison, and a depth-weighted (DW) case with cut-off at 10m is considered as well.

For combinations of $D_s$, $D_v$, and $D_e$ following the Bray method, the depth considered for $D_v$ and the level of conservatism applied to $D_e$ from case histories make a marked difference on total settlement. To capture this variation, three combinations have been proposed, for further comparison and discussion in the next section:

- $D_s + D_v$ considering the entire 20m soil profile
- $D_s + D_v$ considering the top 10m of the soil profile
- $D_s$ considering the top 10m of the soil profile + $D_v$ refined with the Geyin & Maurer (2019) depth weighting (DW) function + $D_e$ as estimated according to case histories discussed previously

## 6 LIQUEFACTION-INDUCED SETTLEMENT COMPARISON AND BEST ESTIMATE

Results from the three methods employed to estimate building-induced seismic settlement are presented in Figure 3 for comparison. Intermediate values for $D_v$, $D_e$, and $D_s$ are shown first including the varying evaluation depths: 20m, 10m, and DW 10m (as considered by Geyin & Maurer, 2019). The Bullock method ($50^{th}$ percentile) gave the lowest settlement estimates, while the Bray method ($50^{th}$ percentile) combination to 20m depth resulted in the largest settlement estimate.

The adopted 'best estimate' for design was a Bray method combination assuming depths exceeding 10m did not contribute to the surface settlement. Remaining over-prediction may also crudely account for the contribution of ejecta-related settlement which isn't explicitly included in this combination due to difficulty quantifying independently with any accuracy. The best estimate values following this method are in the range of both the observed performance in the 2010-2011 CES and the expected performance according to *LPI* and *LSN* criteria.

Table 5 compiles calculated values from the 'Bray' and 'Bullock' methods as a percentage of the representative settlements during CES events. The settlement estimates are an average of data across the study site. The ranges from LiDAR measurements of the CES events are very coarse (+/-100mm) and judgement is required to compare settlement predictions with case history data. Calculated values of *LPI* and *LSN* for the site, as well as observed liquefaction severity, further provide indicative information on the expected site performance, though recent studies have shown *LPI* and *LSN* may not correlate well to observed ejecta severity (Hutabarat and Bray 2021) and should be investigated further.

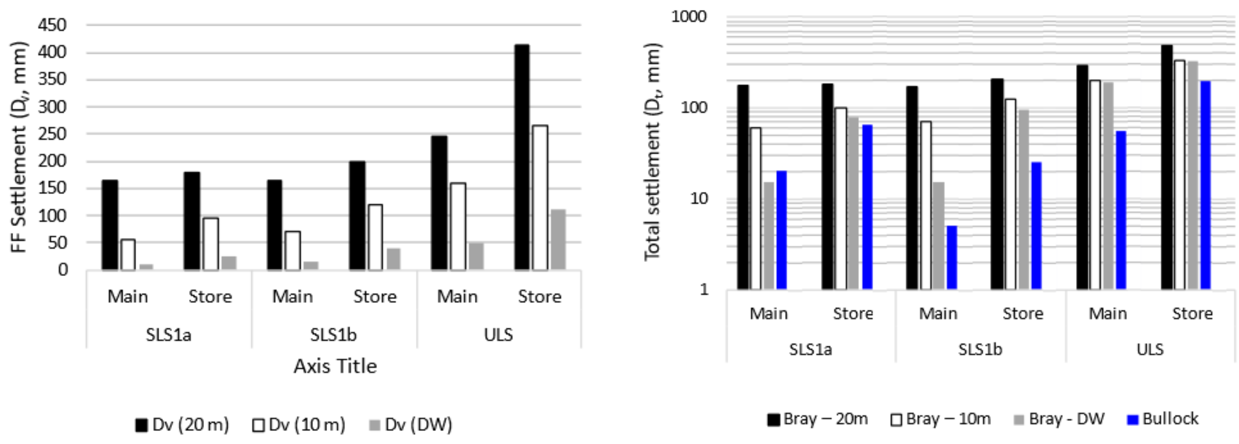


*Figure 3: Summary of free-field (FF) settlements (left) and total settlement (right, note the log scale).*

*Table 5: Estimated total settlements as a percentage of estimated vertical settlements during the CES*

| **FF Settlement Performance during CES** | **SLS1a** | **SLS1b** | **ULS** | **Average** |
|---|---|---|---|---|
| Comparable CES Event | 4-Sep-2010 | 13-Jun-2011 | 22-Feb-2022 | |
| Est. from LiDAR data (mm) | 0 - 200 | 0 - 200 | 100 - 300 | |
| Est. from *LPI* and *LSN* (mm) | 0 - 100 | 100 - 200 | 200+ | |
| Representative CES Performance, $D_t$ (mm) | 50 | 100 | 200 | |
| **Settlement Prediction by Method, $D_t$ (% of CES)** | | | | |
| Bray (FF 20m) | 240% | 200% | 112% | 203% |
| Bray (FF 10m) | 90% | 120% | 83% | 112% |
| Bray (FF DW) | 30% | 100% | 77% | 82% |
| Bullock | 50% | 10% | 25% | 33% |

## 7 CONCLUSIONS

In engineering practice, it is desirable from an economic standpoint to found lightweight small-scale structures on shallow foundations, even with the hazard presented by shallow liquefiable soils. In evaluating the feasibility of doing so it becomes critical to assess the ground and foundation performance to ensure it will reasonably meet code requirements and client performance objectives on a performance basis without overengineering. This is hampered by the large uncertainties in making settlement predictions for future earthquakes using the available simplified empirical and analytical methods, and the crudeness of geotechnical investigation methods, particularly the SPT test undertaken at 1.5m centres downhole.

In this study we have investigated several approaches for estimating liquefaction-induced settlements using available SPT data from the site, with the aim of gauging where the methods may over- or underestimate performance, to inform the selection of a 'best estimate' for design, and from there a suitable range of uncertainty with which to conduct sensitivity studies. Documented past performance of the site during recent earthquakes (i.e., the 2010-2011 CES) provides useful information to inform the likely future performance of the site under moderate and strong design levels of shaking (i.e., SLS, ULS respectively), and reviewing the predictions made by the published methods and the associated assumptions. These advantages will not be present at sites in other parts of the country, but our findings may yield insights into the range of issues with the available methods.

In this comparison, none of the recently developed methods stands out as significantly better than the others. The uncertainty in the Bray model predictions is dominated by large uncertainties in the assumptions made when applying conventional free-field settlement prediction methods, and as yet, no specific method to reasonably quantify the additional settlement due to sand ejecta has been developed and remains a work in progress. The inherent tendency for free field settlement methods to result in over-predictions on account of implicit model biases and correcting for these by limiting to an arbitrary depth and/or applying depth weighting is recommended in developing a 'best-estimate'. The crudeness of the SPT test in undertaking liquefaction assessments and assessing performance may further contribute to a tendency to over-estimate settlements, and the CPT remains the preferred tool for site-specific geotechnical assessments.

From this study we conclude that by limiting the depth of assessment for the free-field component, the Bray method provides settlement estimates more aligned with historic site performance, and that accounting for ejecta-induced settlement introduces more uncertainty than it aims to address. Therefore, further research is required on how best to consider ejecta – possibly through a new semi-empirical free-field settlement prediction model that incorporates ejecta potential inherently.

By contrast the Bullock model, which considers all components together in the one model appears un-conservative in its settlement estimates and consistently forms a lower-bound estimate in our comparison. At present we consider that relying solely on the median estimates from this model as a basis for establishing a 'best estimate' prediction is not recommended.

The rule-of-thumb that settlement predictions should consider a 50 - 200% range on the best estimate is not challenged by our review and retaining a healthy conservatism in developing recommended range for sensitivity assessment is prudent. The challenge presently is understanding where to pitch the 'best estimate' to centre this range, and the study provides some practical recommendations in this regard.

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