September 2013

# Pile design options for shallow depths of liquefaction

## Supplementary guidance to 'Guidance on repairing and rebuilding houses affected by the Canterbury earthquakes', December 2012.

### Simplified procedure for assessing kinematic pile strains for TC3 residential sites in Christchurch

The purpose of this procedure is to enable a simplified analysis of lateral spreading and kinematic interactions to check the suitability of deep pile solutions for domestic dwellings for TC3 sites in Christchurch. A range of pre-engineered deep pile solutions have been checked already using this procedure and these are given in Table 15.3 in Part C of the 'Guidance on repairing and rebuilding houses affected by the Canterbury earthquakes, 2012' (Guidance, 2012). The range of solutions in Table 15.3 is necessarily limited and cannot cover every permutation of pile type and ground condition. For situations not covered in Table 15.3, it is necessary to carry out a specific analysis using the following procedure.

This procedure is a simplified version of the pseudo-static analysis procedure for piles subject to lateral spreading proposed by Cubrinovski et. al. [2009]. The simplifications made here are considered appropriate for the intended and specific purpose of designing deep pile foundations for domestic dwellings in Christchurch. Only a limited range of situations, soil types, and pile types are considered. The following key simplifications and assumptions are made:

Key simplifying assumptions:

- inertial loads from dwelling are modest (or non-existent for sliding pile head detail)
- pile strength/stiffness does not affect the free-field ground deformations
- pile group effects are not significant
- reduced range of soil parameters is considered (but parametric study could be carried out at the discretion of the engineer).

Readers wishing to obtain more background on the subject of kinematic pile interaction and the basis of the procedure should refer to the source paper. (The reference is provided at the end of this document).

The key steps in the simplified procedure are as follows (refer to Figure 1):

Step 1. Formulate ground model

Step 2. Estimate free-field ground deformation

Step 3. Estimate soil-spring parameters

Step 4. Estimate pile moment-curvature relationship

Step 5. Numerical analysis

Step 6. Assess results of analysis

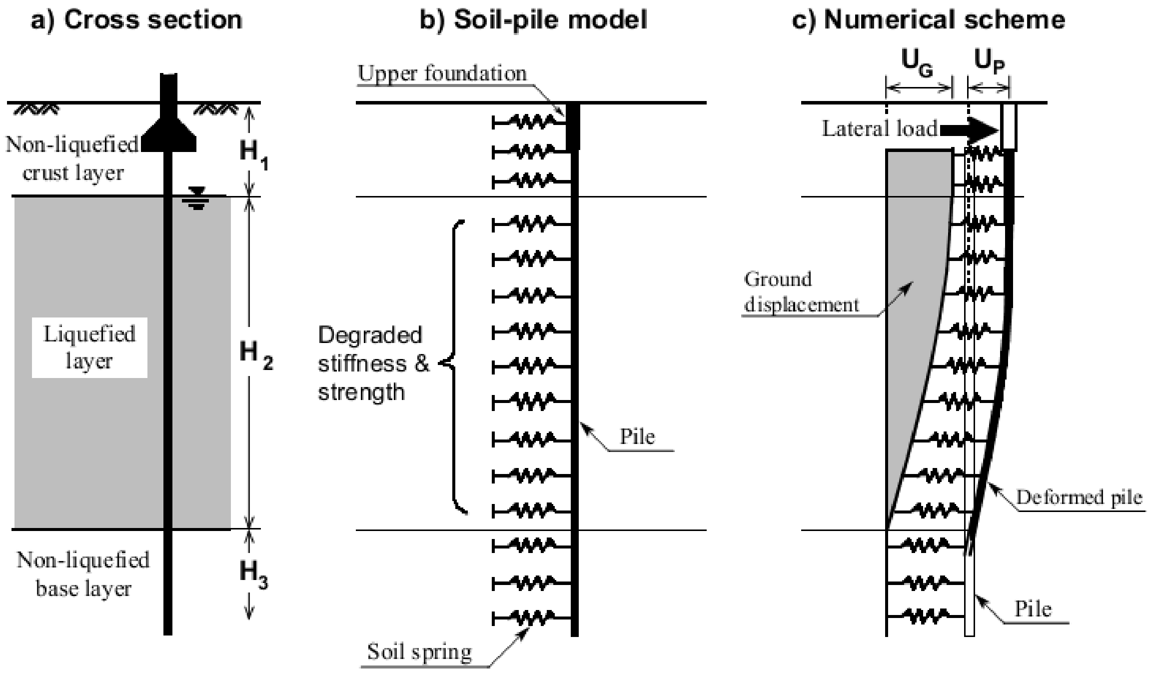



Figure 1. Pseudo-static analysis procedure for piles with lateral spreading.

[Source: Cubrinovski et.al. 2009].

## Step 1. Formulate ground model

As stated in the 'Guidance, 2012' where deep piles are being considered at a site there must be a clearly identifiable bearing stratum that will not liquefy and that will provide adequate support for the pile type being considered (eg, dense sand or gravel with SPT $N_{60}$ > 25 or CPT $q_c$ >15 MPa.). There must be confidence that the bearing stratum is sufficiently thick to provide adequate support for the piles and to bridge over any underlying liquefiable layers (ie, minimum proven thickness of 3 m for single storey and 4m for two storey construction). The bearing stratum must be extensive enough across the site to provide uniform support to the entire footprint of the dwelling.

To prove these requirements, it will be necessary to carry out a deep site investigation (refer to the Guidance). One objective is to identify a suitable bearing stratum with the minimum characteristics identified above. In addition, it is necessary to identify the thickness of the surface crust and other non-liquefying layers to be able to assess the effects of kinematic interactions.

Another objective of the investigation is to develop a ground model for the site similar to that shown in Figure 1 (a). The base of the pile is assumed to be driven (or screwed or bored) into a dense, non-liquefiable bearing layer. The head of the piles is assumed to be within a non-liquefiable crust layer which may be loose/soft or dense/firm of known thickness. Between these two layers is material that is presumed to be liquefiable, in the simplest case, or multi-layered in more complex cases.

More information on requirements for the site investigation is given in the Guidance.

## Step 2. Estimate free-field ground deformation

The free-field ground deformation is the movement of the ground during and after the earthquake excluding any interaction with the embedded piles. In the event of liquefaction of any of the identified layers in the ground model, the surface crust will move relative to the non-liquefied base layer including oscillations during the earthquake and permanent lateral movement after the earthquake. The estimation of these movements is a very complex issue. For the purpose of this simplified procedure, for the Christchurch residential rebuild, the following assumptions may be made consistent with the TC3 Guidance, 2012:

Key assumption: Assume that the maximum displacement of the ground surface relative to the non-liquefied base layer is 300 mm (the minimum requirement stated in the Guidance, 2012. Where greater lateral movements are expected, the Guidance, 2012 recommends that deep pile foundations may not be suitable.

The distribution of the ground surface displacement versus depth is also critical to estimating the strains induced in the pile. Following the recommendation given by Cubrinovski et. al. [2009] assume that all of the ground surface displacement is accommodated within the liquefied layer using a parabolic distribution (see Figure 1 (c) ), with the non-liquefied surface crust moving as a rigid body and the non-liquefied base layer not moving at all.

Comment: A linear distribution of lateral movement through the liquefied layer is an acceptable simplification, eg, see Figure 2.

Where there are thin, intermediate, non-liquefiable layers these may be ignored and treated as part of the liquefied layer. Where there are thick, intermediate, non-liquefiable layers it may be more sensible (using judgement) to consider a multi-layered system where there are two or more liquefiable layers separated by a non-liquefiable layer and distribute part of the ground surface displacement through each liquefiable layer (eg, Figure 2).

Where there are multiple liquefiable layers but with one layer clearly identified as being weaker and more readily liquefiable than the other layers, it may be more sensible and conservative (using judgement) to consider the possibility that all of the ground surface displacement will be concentrated through the weakest layer.

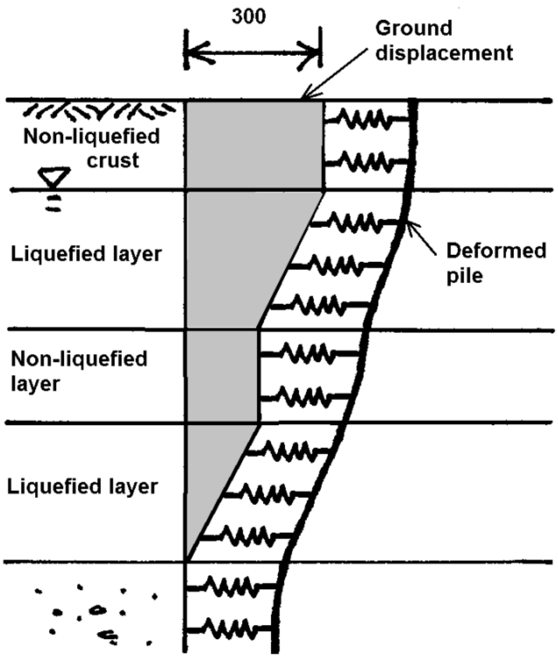
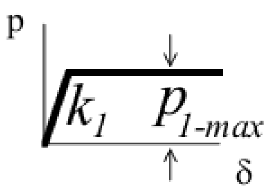



Figure 2. Example of free-field ground displacement for two liquefied layers separated by a non-liquefied layer.

## Step 3. Estimate soil spring parameters

The interaction between the deforming ground and the embedded pile is analysed numerically using Winkler soil-springs. Simple linear elastic-plastic springs may be used with parameters estimated as follows:

<u>Non-liquefied soils</u>:




The soil spring p-y curves for the non-liquefied soil layers may be estimated using empirical procedures developed for static lateral pile loading. The soil spring stiffness is given by:

$$k_i = k_o\, s\, D_o$$

in which $k_i$ = spring constant, $k_o$ = coefficient of subgrade reaction, $s$ = spring spacing, $D_o$ = pile diameter (width). Some suggestions for evaluating $k_o$ are given in the Appendix. The yield strength of the soil spring is given by one of the following:

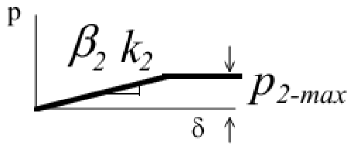
$p_{i\text{-}max} = 4.5\, P_p\, s\, D_o$ (for non-cohesive soils, surface crust)

$p_{i\text{-}max} = 3\, P_p\, s\, D_o$ (for non-cohesive soils, deeper layers)

$p_{i\text{-}max} = 9\, S_u\, s\, D_o$ (for cohesive soils)

in which $p_{i\text{-}max}$ = yield strength of soil spring, $P_p$ = Rankine passive pressure, $S_u$ = undrained shear strength, $s$ = spring spacing, $D_o$ = pile diameter (width).

Liquefied soils:




The soil spring p-y curves for the liquefied soil layers may be estimated using the same procedures as for the non-liquefied soils but with the following modifications:

The soil spring stiffness is reduced by stiffness degradation factor $\beta_2$ after liquefaction, which may be taken as 0.01. The soil spring yield force is given by:

$$p_{i\text{-}max} = S_r \, s \, D_o \qquad \text{(for liquefied soils)}$$

in which $S_r$ = residual strength of the liquefied soil layer. For the purpose of this procedure for residential dwellings in Christchurch assume that $S_r$ is 5 KPa, with a likely range between 5 KPa and 15 KPa.[1]

## Step 4. Estimate pile moment-curvature relationship

Estimating the pile moment-curvature relationship is a key step. Deep piles at sites with lateral spreading may be expected to suffer significant curvatures and plastic hinge formation near the interfaces between liquefying and non-liquefying layers. Yielding and hinge formation are acceptable up to certain strain limits, established to ensure that the axial load carrying capacity of the piles is not compromised. The pile moment-curvature relationship needs to be realistic and extend well into the plastic range if the analysis is to provide useful predictions of pile strains.

Comment: Simple assumptions of elastic response and first yield moment limits will result in very conservative pile designs.

Procedures for calculating moment-curvature relationships for reinforced concrete, pre-stressed concrete and steel sections are given by Priestley et. al.[2007].

Reinforced concrete and pre-stressed concrete piles need very careful detailing to achieve acceptable ductility and moment-curvature performance. Examples of moment-curvature relationships for two commercially available pre-stressed concrete piles suitable for residential housing in Christchurch are given in Figure 3[2].

The moment-curvature relationship depends significantly on the simultaneous axial loading in the pile. Moment-curvature relationships need to be developed to bracket the range of pile axial loading expected at the time of the earthquake (i.e. maximum and minimum axial loads).

Limiting values of pile curvature should be established in each case based on recommended extreme fibre strain limits given in Table 1.

1 For a more rigorous treatment of $S_r$ refer to Idriss and Boulanger [2008].
2 Details in figure 3 provided by Hi-Stress Limited, Christchurch

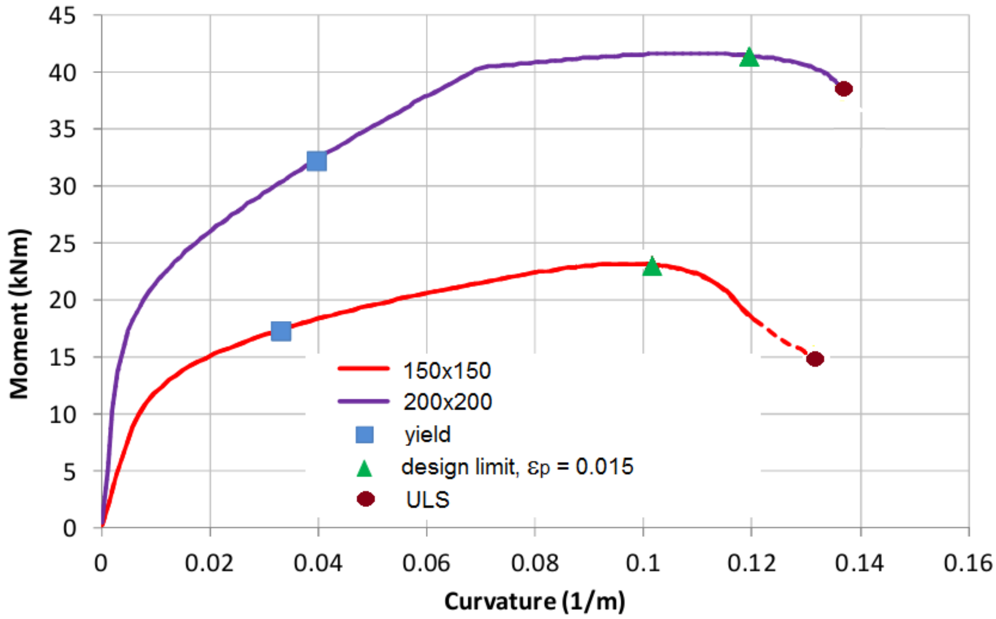



Figure 3. Example moment-curvature[3] relationship for small pre-stressed concrete piles (with 100 KN axial load).

Table 1. In-ground plastic hinge strain limits for residential piles.[4]

| Pile Type | Strain Limits |
|---|---|
| Pre-stressed solid concrete piles | $\varepsilon_c \leq 0.008$ ($\varepsilon_c \leq 0.005$)* $\varepsilon_p \leq 0.015$ |
| Steel pipe piles | $\varepsilon_s \leq 0.010$ |
| Steel pipe piles (concrete filled) | $\varepsilon_s \leq 0.010$ |
| Timber piles (normal and high density) | $\varepsilon_t \leq 0.0034$ |

$\varepsilon_c$ = extreme fibre concrete compressive strain

$\varepsilon_p$ = pre-stressing strand tensile strain

$\varepsilon_s$ = steel shell extreme fibre strain

$\varepsilon_t$ = timber extreme fibre strain

*Note that peak curvature will always develop in the more competent crust or bearing layer where the surrounding soil confines the compression face of the concrete pile permitting a higher strain limit of $\varepsilon_c \leq 0.008$. However, within the liquefied layer the degree of confinement will be minimal and a reduced strain limit of $\varepsilon_c \leq 0.005$ is recommended.

[3] Curvature, $\kappa = 1/R$ = inverse of the radius of curvature (R) at any point on the pile and has dimension 1/length. For an elastic pile, the bending moment at any point is proportional to the curvature at that point, $M = \kappa EI$. Most piles will go beyond the elastic limit and so it is necessary to obtain the non-linear relationship between Bending Moment and Curvature, as shown in Figure 3 for example. (E = modulus of elasticity of pile material, I = moment of inertia of pile cross-section).

[4] Adapted from POLA [2010], for Operational Level Earthquake (OLE, 75 year return period), i.e. conservative for Ultimate Limit State (ULS) case and should permit re-use after both Service Limit State (SLS) and ULS earthquake.

For timber piles, a linear relationship should be assumed using the following values for bending elastic modulus based on NZS 3603:1992:

$E_t$ = 11,000 MPa (high density piles in wet condition, steamed and machine shaved radiata logs)

$E_t$ = 8,000 MPa (normal density piles in wet condition, steamed and machine shaved radiata logs)

Limiting values of curvature for timber piles are given in Table 2.

Table 2. Limiting curvature for timber piles.

| Pile SED (mm) | Limiting Curvature (1/m) |
|---|---|
| 250 | 0.028 |
| 275 | 0.025 |
| 300 | 0.023 |
| 325 | 0.022 |
| 350 | 0.020 |

## Step 5. Numerical analysis

The Winkler spring analysis should be carried out using suitable software with the following features:

- Spring spacing should be preferably 0.1 m and not larger than 0.2 m
- Elastic-plastic soil spring model
- Bi-linear pile model (tri-linear preferred for reinforced concrete piles)
- Estimated free-field ground displacement as input
- Output including pile curvature versus depth (or bending moment versus depth in format able to be converted to curvature versus depth)

Commonly available commercial software with the above capabilities includes the following (other suitable software may be available):

- ***Lpile*** published by Ensoft, Inc at; http://www.ensoftinc.com/
- ***Seismostruct*** published by Seismosoft Ltd at; http://www.seismosoft.com/en/SeismoStruct.aspx

Comment: L-Pile provides the option of either inputting user defined p-y curves or using inbuilt p-y curves which are different to those recommended in this document. The Inbuilt p-y curves appear to give a more conservative calculation of pile curvature, based on a limited case study comparing outputs.

The end fixity conditions of the pile should be set realistically: For most residential dwellings the pile head is unlikely to achieve structural fixity. For the standard sliding head detail given in the Guidance, 2012, the pile head should be considered as pinned. The pile head may achieve effective fixity within the soil crust, but this will be determined by the Winkler spring analysis.

## Step 6. Assess results of analysis

The key output from the Winkler spring analysis should be a plot of pile curvature versus depth along the full length of the pile. A plot of pile displacement versus depth should also be obtained as a useful "reality" check of the analysis (e.g. see Figure 4).

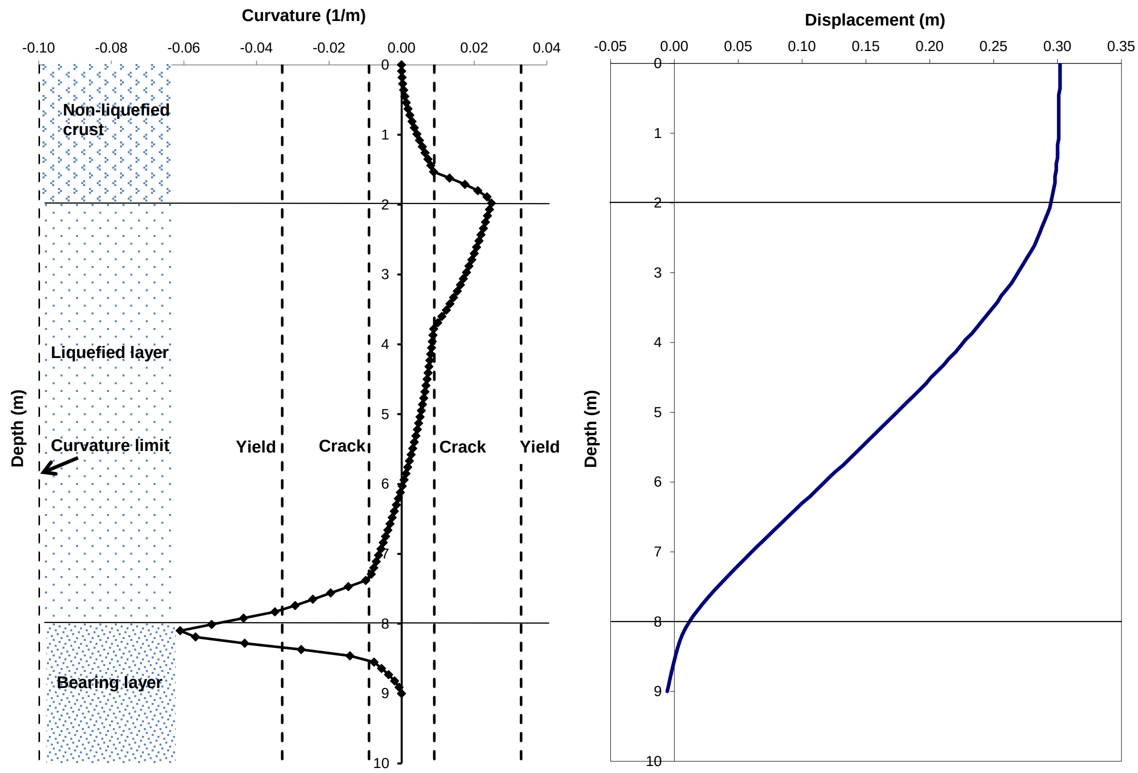



Figure 4. Curvature and displacement versus depth for 150x150 pre-stressed concrete pile at a site with 6m thick liquefied layer.

The peak values of pile curvature will be located near to critical interfaces in the ground model, i.e. the interfaces between the assumed liquefied and non-liquefied layers. In the example shown in Figure 4, the maximum pile curvature was found to be at the interface between the liquefied layer and the much stiffer bearing layer. Another, lesser peak in curvature was found at the interface between the non-liquefied crust and the underlying liquefied layer. The maximum curvature in this example (0.06 1/m) exceeded the concrete cracking strain and the yield point of the pre-stressing strand, but was well within the design limit for the pile (0.1 1/m).

If the maximum pile curvature is found to exceed the design limit for the pile then the pile should be considered unsuitable and a different pile section trialled. In general, more flexible and ductile piles will work better than stiffer or more brittle piles.

## P-Δ Effects

The piles are required only to provide vertical support to the dwelling and are not intended to provide any lateral resistance to the ground movement. For most cases, lateral instability of the piles is prevented by the surface crust and P-Δ effects do not need to be considered separately.

For those rare cases where the water table is at the ground surface and where liquefaction may extend to the ground surface without any significant crust, then the effect of P-Δ moments should be considered and should not exceed 50% of the design limit moment capacity of the piles.

## References:

Cubrinovski, M., Ishihara, K., and Poluos, H.J. (2009). "Pseudo-static analysis of piles subjected to lateral spreading," *Bulletin of the N.Z.N.S.E.E.,* Vol. 42, No. 1, March 2009, pp. 28-37.

Idriss, I.M. and Boulanger, R.W. (2008). "Soil Liquefaction during Earthquakes," EERI publication number MNO-12, 237 p

POLA (2010). "The Port of Los Angeles Code for Seismic Design, Upgrade and Repair of Container Wharves," City of Los Angeles Harbor Department, May 2010

## Appendix

### *Suggestion for estimating soil coefficient of subgrade reaction for laterally loaded piles:*

Empirical expression based on the SPT blow count:

$$k_o = 56N(100D_o)^{-0.75}$$

in which $k_o$ = coefficient of subgrade reaction in MN/m$^3$, $N$ = representative SPT blow count for the soil layer, $D_o$ = pile diameter in m. Note that the representative value of $N$ for the base layer should be the average over the depth of embedment of the pile into the layer.

[From Japanese Design Code for Building Foundations, AIJ, 2001]

Where CPT data is available instead of SPT data, the equivalent SPT blow count, $N$, can be estimated using:

$$N \approx 2.5\, q_c$$

in which $q_c$ = CPT tip resistance in MN/m$^2$ (applies to sandy soils).