Liquefaction induced kinematic loads on piles and inertia loads - literature review and design suggestions

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ABSTRACT

Earthquake shaking can cause liquefaction in certain types of soil. The associated loss of soil strength and stiffness coupled with any lateral ground movement can impart large lateral loads (kinematic loads) to buildings and their foundations, which can be very damaging. In addition to the kinematic loads, the foundations are also subjected to lateral inertia loading (base shear) from the building. Inadequate consideration of these loads can result in unacceptable performance of the foundation and structure. Various methods to assess the components of kinematic load for pseudo-static analyses are available. This paper presents a literature review of these methods. It was found that calculated kinematic loads on a pile can vary significantly depending on the method chosen, and soil profile present. Comparisons are made for an example soil profile, and suggestions provided for design. This paper also presents a literature review of recommended combinations of concurrent base shear and kinematic loads. It was found that there is significant variability between guidelines, and the majority of the guidelines are typically for low period structures (bridges, wharves etc). Suggestions are provided on percentages of peak base shear to consider in combination with kinematic loads, and factors to consider when making this selection.

1 INTRODUCTION

Earthquake shaking can cause liquefaction in certain types of soil. The associated loss of soil strength and stiffness coupled with any lateral ground movement can be damaging to buildings and their foundations. Pile foundations in liquefied ground can be subject to kinematic (soil) loads resulting from ground lurch/cyclic displacement (during the earthquake shaking), and lateral spread (typically towards the end of earthquake shaking or post-earthquake). There are three main components of kinematic load on the foundations resulting from such ground movement. 1) the non-liquefied soil load (crust) on the pile cap and other buried elements; 2) the crust load on pile; and 3) liquefied soil load on pile. In addition to the kinematic loads, the foundations are also subjected to lateral inertia loading (base shear) from the building. Inadequate consideration of these loads can result in unacceptable performance of the foundation and structure. Various methods to assess the components of kinematic load for pseudo-static analyses are available. Section 2 presents a literature review of these methods. Comparisons are made between methods of the loads applied to a pile and the resulting actions for an example soil profile. Suggestions are provided for design.

Structures subject to earthquake shaking will also impose inertia (base shear) loads on piles. Section 3 presents a literature review of recommended combinations of concurrent inertia and kinematic loads.

Suggestions are provided on percentages of peak base shear to consider in combination with kinematic loads during the four phases of an earthquake (no liquefaction; after liquefaction triggering (no kinematic loads); cyclic displacement and lateral spread), and factors to consider when making this selection.

2 KINEMATIC LOADS

Historic earthquakes have shown that most piles suffered the largest damage at the pile head and in the zone of the interface between the liquefied layer and the underlying non-liquefied base layer. Damage at the pile head was due to both inertial loads from the structure, and kinematic loads due to the lateral ground displacement. Damage at the interface of the liquefied layer and base layer was due to the kinematic loads arising from large ground movements and the large stiffness contrast of the two layers (Cubrinovski & Ishihara 2004 and Cubrinovski et al 2009).

The following subsections compares conventional methods for assessing kinematic loads, as well as those outlined in two commonly used guidelines in New Zealand (Cubrinovski et al 2014 and Ashford et al 2011). These guidelines have been specifically developed for design of pile foundations for bridges in laterally spreading ground.

2.1 Non-liquefied soil (crust) load on substructure

In laterally displacing ground (i.e. cyclic displacement and lateral spread scenarios), the non-liquefied soil (crust) can impart a load on the substructure (pile cap, foundation beam, basement wall etc). The ultimate lateral load imparted will be the passive pressure plus friction on the sides and base. The passive pressure is usually the main component of this load. A comparison of methods used to assess the passive load is presented in Table 1. The load can be calculated by equation 1.

$$P_P = \sigma'_{v\text{-}ave} . K_p . H \tag{1}$$

where P_P = passive earth load on the pile cap, foundation beam or wall (substructure); $\sigma'_{v\text{-}ave}$ = average effective stress over the height of the substructure; K_P = passive earth pressure coefficient; H = height.

Table 1: Non-liquefied soil (crust) load on substructure

Method	**Description**
Rankine[1]	Conventional method using Rankine K_p (planar failure surface / no wall friction).
Coulomb[1]	Conventional method using Coulomb K_p (log-spiral failure surface / with wall fiction). For passive loading a wall friction coefficient, δ/ϕ = 0.5 is typically adopted.
Ashford et al (2011)	Notes wall friction may be significantly lower when a crust spreads against a wall than when a wall is pushed into a non-liquefied soil because: • the spreading crust may settle due to extensional strains, cracking, and sand boil formation, and this settlement will negate the formation of upward directional stresses on the back of the wall; and • the underlying liquefied sand provides a soft and weak boundary condition on the base of the deposit that permits lateral stresses to spread a large distance upslope

Method	Description
	of from the wall, and the resulting failure mechanism is associated more closely with Ranking earth pressure theory than Coulomb / log-spiral. States that until further research is available to clarify appropriate selection of wall friction parameters for lateral spreading, the friction should be reduced by half from the value that would be used for a non-liquefied soil profile and the earth pressure computed using log-spiral theory. It is assumed that this corresponds to a $\delta/\phi = 0.25$.
Cubrinovski et al (2014)	Document does not implicitly state value to use, however based on text it is inferred that the Rankine method is to be applied.

Note 1: Conventional method for calculation of passive earth pressures. Not specifically developed for determination of passive load on structure for a case with liquefied soils.

2.2 Crust load on pile

As for the crust load acting on the substructure, the non-liquefied soil will also impose a lateral load on the pile. Methods to assess this load are presented in Table 2.

Table 2: Non-liquefied soil (crust) load on pile

Method	Description [2]
(Broms 1964a) for granular soil [1]	• $P_{c\text{-}max} = \alpha_c.P_p.D.s$ [3] • Experimental test data indicated α_c typically in the range of 3 to 6. 3 is adopted as conservative value for active piles [2]. A higher value is likely to be appropriate in design for passive loading [2]. P_p is the Rankine passive pressure.
(Reese et al 1974) p-y model for Sand or API Sand (API 2011) p-y model [1]	• $P_{c\text{-}max} = \min(P_{cs}, P_{cd})$ [3] where: Ultimate lateral crust load (shallow failure), $P_{cs} = (C_1.z + C_2.D).\sigma'_v.s$ Ultimate lateral crust load (deep failure), $P_{cd} = C_3.D.\sigma'_v.s$ C_1, C_2 and C_3 are coefficients based on the friction angle. Note this method was developed for active piles [2] where the minimum of the shallow and deep failure mode capacity is adopted. This can result in the transition from shallow to deep failure capacity at significant depth. For passive piles, the designer should assess whether the transition depth is appropriate.
(Ashford et al 2011)	• Use conventional method to assess load (e.g. (Reese et al 1974) or API Sand). • Consider smeared profile, refer Table 4 below (Ashford et al 2011) method). • Do not consider group effects (unconservative reduction in forces). If piles are closely spaced (i.e. act as a wall); modify so total load equals that on a wall.
(Cubrinovski et al 2014)	• Use (Broms 1964a) granular soil method, however consider a reference α_c value of 4.5. • A lower and upper bound α_c value of 3 and 5 are suggested for sensitivity analyses. It is noted that the α_c value of 3 suggested by (Broms 1964a) may be unconservative for passive piles [2].

Notes:

1. Conventional methods for assessment of lateral capacity. Not specifically developed for determination of passive load on piles for a case with laterally moving soils.

2. Active piles are where the pile pushes into the ground. Passive piles are where ground pushes into the pile.

3. Where $P_{c\text{-}max}$ is the ultimate lateral crust load (kN) on pile at a depth below ground level. α_c is the scaling factor to account for difference between lateral pressure on a pile vs continuous wall. D is the diameter of the pile. σ'_v is the effective stress at depth below ground level. z is depth below ground level. s is spring spacing.

2.3 Liquefied soil load on pile

Liquefied soil is generally modelled as a clay with a liquefied shear strength, Sr. Various published methods are available to assess Sr, and are typically based on the soils pre-liquefied relative density and effective stress. Alternatively, liquefied soil is modelled by applying a p-multiplier to the soils pre-liquefied strength. Four published methods for assessment of liquefied passive soil load on a pile are presented in Table 3.

Table 3: Liquefied soil load on pile

Method	Description (2)
(Ashford et al 2011) p-multiplier method	• Applies a p-multiplier, m_p, to the pre-liquefied soil p-y curve (as assessed using a conventional method). • The p-multiplier modifies both the strength and stiffness of the soil. The multiplier varies depending on the soils relative density. E.g. for an SPT $(N1)_{60\text{-}CS}$ of 8 to 16 an m_p range of 0.05 to 0.2 is suggested. • It is suggested that performance evaluations/design use p-multipliers in the middle of this range. Sensitivity of the expected foundation performance to a factor of 2 increases and decreases in m_p should be evaluated.
(Ashford et al 2011) Sr method	• Uses the soils estimated liquefied shear strength, Sr, with a relation appropriate for undrained behaviour of clay. E.g. (Matlock 1970) p-y model. I.e. $P_{L\text{-}max} = \min(P_{Ls}, P_{Ld})$ (2) where: Ultimate liquefied soil load (shallow failure), $P_{Ls} = 3.S_r.D.s + \sigma'_v.D.s + J.S_r.z.s$ (1) Ultimate liquefied soil load (deep failure), $P_{Ld} = 9.S_r.D.s$ Note model was developed for active piles (2) where the minimum of the shallow and deep failure mode capacity is used. This can result in the transition from shallow to deep failure capacity at significant depth. For passive piles, the designer should assess whether the transition depth is appropriate.
(Cubrinovski et al 2014)	• Uses soils liquefied shear strength, S_r, and (Broms 1964b) cohesive soil method for calculation of the ultimate lateral capacity ($P_{L\text{-}max} = \alpha_L.S_r.D.s$), but with a modified α value as set out below. • α_L value of 1 is recommended as the lower bound and reference value. An $\alpha_L > 1$ could be considered in parametric evaluations. • The stiffness is assessed using the method suggested for crust soils, but with a stiffness degradation factor, β_L, applied. The reference values for β_L for the cyclic displacement and lateral spreading scenarios are 0.05 and 0.01 respectively.

Method	Description [2]
(Franke and Rollins 2013)	• Recommends use of a hybrid p-y model comprising the lesser (lower-bound envelope) of the (Rollins et al. 2005a) and (Wang and Reese 1998) p-y curves. • The (Rollins et al 2005) model was developed to quantify the dilative p-y behaviour of liquefied sand (but can produce high soil resistance if used outside the bounds the model was calibrated). The (Wang and Reese 1998) model, which uses the (Matlock 1970) model in conjunction with the liquefied soil strength, is used to define the limiting soil strength. (Franke and Rollins 2013) recommends the 33rd-percentile residual strengths from (Seed and Harder 1990) are used. Also recommends an equation to modify the strain parameter to account for the stiffer load-resistance behaviour of denser liquefiable soils. • Recommends that zero strength be assigned for liquefied soils with $(N_1)_{60\text{-}cs} < 5$ ($D_r <$ 30%) until additional case histories can justify higher values.

Notes

1: J is a dimensionless empirical constant with values ranging from 0.25 to 0.5.

2. $P_{L\text{-}max}$ is the ultimate lateral load (kN) on pile from the liquefied soil at a depth below ground level.

2.4 Non-liquefied base layer

The pile extent below the liquefied layer (i.e. in the non-liquefied base layer) will behave as an active pile. Considerations presented in the (Ashford et al 2011) and (Cubrinovski et al 2014) are outlined in Table 4.

Table 4: Non-liquefied deeper soil

Method	Description [2]
(Ashford et al 2011)	• Use conventional method to assess lateral capacity (e.g. (Reese et al 1974) or API Sand) • Consider smeared profile [1], where the capacity linearly increases from the liquefied layer to the base layer capacity over a distance of 2D, refer Figure 1.
(Cubrinovski et al 2014)	• Use Broms method for granular soil however consider an α_B value as set out below. • Ultimate lateral capacity of base layer, $P_{B\text{-}max} = \alpha_B.P_p.D.s.$ where: $\alpha_B = 3$; however, $\alpha_B = 1$ could also be used and is the preferred choice at larger depths. P_p is the Rankine passive pressure.

Notes:

1. The presence of a liquefied layer will reduce the lateral capacity of the soil immediately above and below (i.e. in the crust and base layer). However, the appropriate distance for smearing against large diameter piles requires further study (because the distance 2D can equal or exceed the thickness of the crust when D is large and the crust is thin); therefore the smeared profile should not be used to reduce the ultimate passive load that a non-liquefied crust can impose on large diameter pile shafts. Where the crust is providing resistance, an appropriate smear profile should be considered.

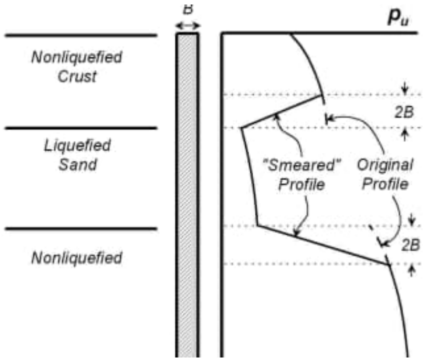

Figure 1: Modification to the profile of ultimate subgrade reaction, p_u, to account for the weakening effect the liquefied sand exerts on overlying and underlying non-liquefied layers (from Ashford et al 2011)

2.5 Comparison of results by method

Mixing of methods is not recommended. The complete analysis should follow one method. Repeating the analysis for a number of methods, sensitivity analyses, critical assessment of the results and application of engineering judgement to select a design conclusion and range is recommended.

For an example soil profile, refer Figure 2, a comparison was carried out on loads calculated and applied to a pile, and the resulting pile actions when adopting the recommendations outlined in (Cubrinovski et al 2014) and (Ashford et al 2011). The results are presented in Table 5 and Table 6, and Figure 3.

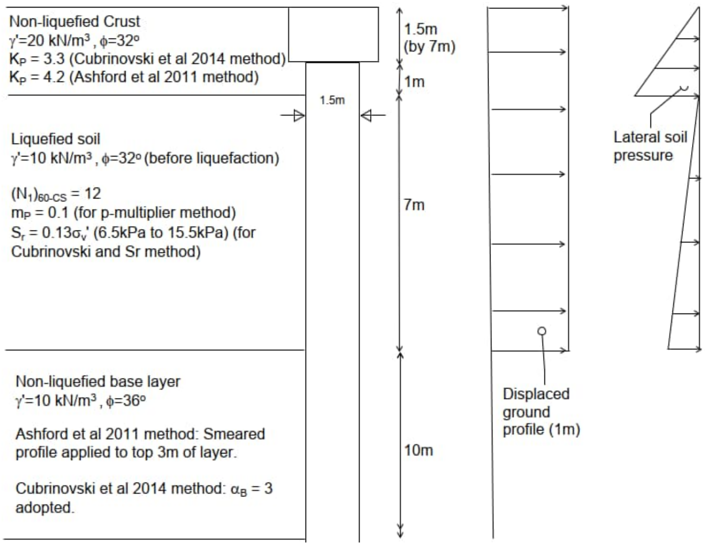

Figure 2: Example soil profile and parameters

Table 5: Comparison of loads by method

Method	Crust load on foundation beam (kN)	Crust load on pile (kN)	Liquefied soil load on pile (kN)
(Ashford et al 2011) p-multiplier method	660	675	1100
(Ashford et al 2011) Sr method	660	675	1050
(Cubrinovski et al 2014)	520	890	115

Table 6: Comparison of pile actions by method

Method	**Moment at top of pile (kNm)**	**Moment at interface of liquefied soil and non-liquefied base layer (kNm)**	**Shear at top of pile (kNm)**	**Shear at interface of liquefied soil and non-liquefied base layer (kNm)**	**Deflection at top of pile (mm)**
(Ashford et al 2011) p-multiplier method	-10,640	7,100	660	2,360	50
(Ashford et al 2011) Sr method	-10,650	7,100	660	2,350	50
(Cubrinovski et al 2014)	-8,650	3,900	520	1,500	35

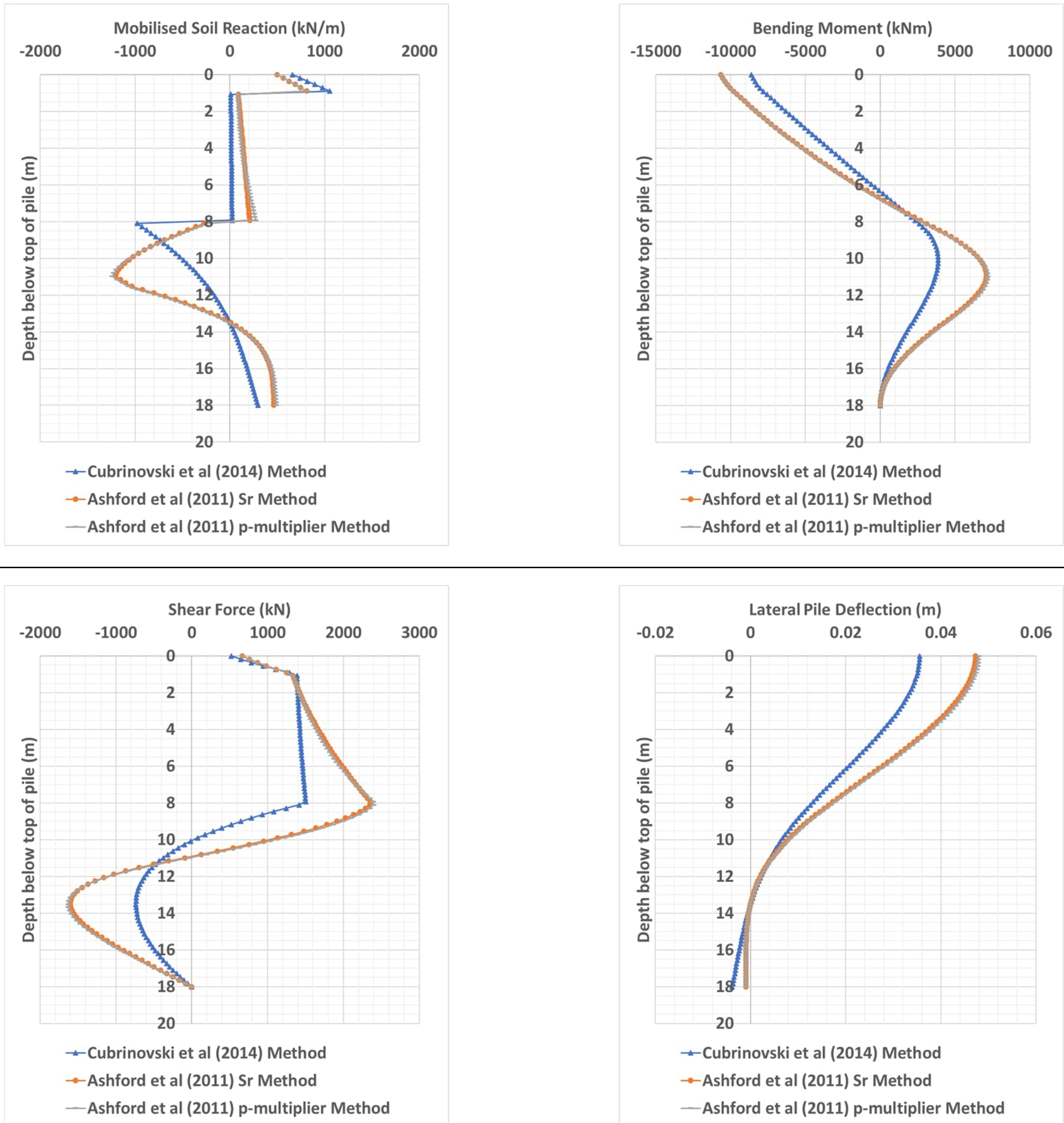

Figure 3: Comparison of pile actions by method

2.6 Discussion and suggestions for design

The below observations are made from the results of the methods compared, as shown in Section 2.5:

- The crust load on the pile cap is higher for the (Ashford et al 2011) method, and the crust load on pile is higher for the (Cubrinovski et al 2014) method.
- The liquefied soil load on the pile is significantly higher for the (Ashford et al 2011) methods.
- The actions in the pile are significantly higher for the (Ashford et al 2011) methods.
- There was no significant difference between the loads or actions in the pile between the (Ashford et al 2011) methods for the chosen example. However, the liquefied soil load on the pile for the respective methods could be sensitive to the soil profile, liquefied soil strength and p-multiplier.
- If the soil profile comprised a thicker crust and thinner liquefied soil layer, it is likely that the (Cubrinovski et al 2014) method would result in larger soil loads and pile actions.

For design, the following is suggested:

- Sensitivity analysis on the design method (e.g. Cubrinovski et al or Ashford et al methods). The method resulting in the largest pile actions should be considered as the base case.
- Sensitivity analysis on the key input parameters for the three methods (e.g. crust thickness, liquefied soil strength, and soil lateral capacity considering whether it is a load or resistance). It is suggested that a high estimate value for the liquefied soil strength be considered for the sensitivity. (Ashford et al 2011) suggests a p-multiplier of two times the base case value be considered for that method.
- The design team agree performance objectives for both the design case and high estimate sensitivity case. For consideration, pile bending strain limits are provided in Module 4 (NZGS & MBIE 2021). ASCE (2014) provides strain limits for various performance objectives / limit states.
- Designers should consider that the degree of liquefaction may not be uniform across the site. For example, one part of the site being only partially liquefied, the associated piles being in stiffer ground and attracting greater shear load. This would be resolved using displacement-based design (assuming all pile heads are tied together).

3 CONCURRENT BASE SHEAR AND KINEMATIC LOADS

Inertia (base shear) load from the structure could occur concurrently with kinematic loads from the soil, but it is unlikely that this inertia load will be the full design inertia load. This is because; a) it is unlikely that inertia and kinematic loads will be perfectly in phase during shaking, b) it is unlikely that the maximum inertia load and the maximum kinematic load will develop at the same time during the sequence of the shaking, and c) development of liquefaction is required to initiate kinematic loading, this liquefaction could cause a spectral period shift and reduction in inertia load. What percentage of peak inertia load to consider in conjunction with kinematic loads requires careful consideration. Recommendations provided in available guidelines are presented in Table 7 below.

Table 7: Comparison of inertia loads to consider by guideline (adapted from (Souri et al 2022))

Guideline	Recommended % of (pre-liquefaction) peak inertia load with kinematic loads
Module 4 (NZGS & MBIE 2021)	Document application: Buildings • No liquefaction: 100% of peak inertia (kinematic 0)

<table>
<thead>
<tr><th>Guideline</th><th>Recommended % of (pre-liquefaction) peak inertia load with kinematic loads</th></tr>
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After liquefaction triggering:

Stiff piles: may continue to attract significant inertial loads in addition to kinematic loads. Some reduction in building dynamic response is likely depending on how much reduction in foundation stiffness occurs following liquefaction.
Flexible/weak piles: the building dynamic response is likely to change significantly because of softening of the foundations: the period will elongate; accelerations will decrease; but displacements may increase. Building inertial loads will still be applied to the piles, but often at a reduced level from peak.
Recommends carrying out parametric study due to uncertainties arising from the unpredictable nature of the ground motion characteristics (time history, and hence temporal evolution of the response) and also from the reduction of a complex dynamic problem to an equivalent static analysis.
Notes that some general guidance for selection of combined forces is given in (Tokimatsu et al 2005) and (Boulanger et al 2007).
(Tokimatsu et al 2005) carried out analyses which showed:
1) If the period of the superstructure is less than the ground, the kinematic and inertial force tends to be in phase, increasing the stress in piles.
2) If the natural period of the superstructure is greater than that of the ground, the kinematic force tends to be out of phase with the inertial force, restraining the pile stress from increasing.

Lateral spreading: Usually lateral spread does not reach its maximum displacement until the end of ground shaking or later, so for most cases it would be safe to assume that simultaneous building inertial loads can be either ignored or significantly reduced, especially for flexible piles.

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<tr><td>Bridge Manual (NZTA 2022)</td><td>
Document application: Highway Bridges

No liquefaction: 100% of peak inertia (kinematic 0)
Cyclic displacement: 80% of peak inertia
Lateral spreading: May be ignored, except where the percentage of the hazard contributing to the peak ground acceleration by a magnitude 7.5 or greater earthquake is more than 20%, consider the plastic hinge force or 25% of the structure inertial forces, whichever is less.

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<tr><td>Boulanger (2007) / Ashford et al (2011)</td><td>
Document application: Highway Bridges
Short relative site to building period ($Sa_{T=1s}$ / $Sa_{T=0s} \leq 0.4$) [(1)]:

No liquefaction: 100% inertia (kinematic 0)
After liquefaction triggering (No kinematic loads): From pile cap = 35% of peak inertia; from superstructure = 45% of peak inertia [(2)]
Lateral spreading: From pile cap and superstructure = 30% of peak inertia [(3)]

Medium relative site to building period (($Sa_{T=1s}$ / $Sa_{T=0s}$ = 0.5 to 1.6) [(1)]:

No liquefaction: 100% of peak inertia (kinematic 0)

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Guideline	Recommended % of (pre-liquefaction) peak inertia load with kinematic loads
	• After liquefaction triggering (No kinematic loads): From pile cap = 75% of peak inertia; from superstructure = 55% of peak inertia (2) • Lateral spreading: From pile cap = 64% of peak inertia; from superstructure = 36% of peak inertia (3) Long relative site to building period ($Sa_{T=1s}$ / $Sa_{T=0s}$ = 1.7 to 2.4) (1): • No liquefaction: 100% of peak inertia • After liquefaction triggering (No kinematic loads): From pile cap = 140% of peak inertia; from superstructure = 75% of peak inertia (2) • Lateral spreading: From pile cap = 119% of peak inertia; from superstructure = 49% of peak inertia (3)
Caltrans (2012) and ODOT (2014)	Document application: Highway Bridges • 100% kinematic and 50% of peak inertia
WSDOT (2021)	Document application: Highway Bridges • 100% kinematic and 25% of peak inertia
AASHTO (2014)	Document application: Highway Bridges • Design piles for the simultaneous effects of inertial and lateral spreading loads only for large-magnitude earthquakes (M > 8).
MCEER (2003)	Document application: Highway Bridges • For most earthquakes, peak inertia is likely to occur early in the ground motion. Design piles for independent effects of inertia and lateral spreading. For large-magnitude and long-duration earthquakes the two loads may interact.
ASCE (2014) and POLB (2015)	Document application: Wharves and piers • Inertia and kinematic loads should be treated uncoupled for marginal wharves (because maximum bending moments occur at different times).
POA (2017)	Document application: Wharves and piers • 100% kinematic and 100% of peak inertia. Smaller factors are allowed if peer-reviewed 2D nonlinear numerical analysis is used (no less than 25%).
Souri et al (2022)	Document application: Wharves and piers The below is applicable for relatively flexible piles with small diameters (up to about 0.7m). The interaction of inertial and kinematic loads could be different for pile shafts with larger diameters.

Guideline	Recommended % of (pre-liquefaction) peak inertia load with kinematic loads
	• Profiles that can be characterised as configurations that include deep seated liquefaction underlying significant non-liquefiable crust: 30% to 60% of peak deck inertial forces as determined for liquefied conditions. • Profiles that can be characterised as configurations that include smaller kinematic demands/loads associated with either non-liquefiable profile or weak/softened soils closer to the ground surface and thin non-liquefiable crust: 90% to 100% of peak deck inertial forces as determined for liquefied conditions. • If peak inertia load for non-liquefied conditions is evaluated, an additional multiplier may be needed (e.g. Boulanger (2007) values presented above for "After liquefaction triggering (No kinematic loads)" scenario).

Notes:

1. Values in (Ashford et al 2011) are based on (Boulanger et al 2007), and were formulated for the case without any restraint at the top of the pier column(s) from the superstructure. Future research required to better quantify the influence of liquefaction when the superstructure does provide restraint. In the absence of better information, the values suggested can be used for the case where columns are restrained. Sa_T is the linear-elastic spectral acceleration (5% damping ratio) at period, T.

2. The values presented consider the change in inertia load due to the soils liquefying. Note this "After liquefaction triggering" scenario is not specifically presented in (Ashford et al 2014). Values are based on interpretation of (Boulanger et al 2007).

3. The values presented consider the change in inertia load due to the soils liquefying. It also considers the fraction of maximum inertial load with liquefaction that occurs at the critical loading cycle (i.e. when the maximum pile bending moments and shear forces occurs).

3.1 Factors for consideration and suggestions for design

A review of the published literature shows there is significant variation in the inertia load recommended to be adopted in combination with kinematic loads. The majority of literature is for low period structures (bridges, wharves etc); only Module 4 (NZGS & MBIE 2021) is for buildings. It is suggested that the following factors should be considered when selecting a % peak inertia load (as determined prior to liquefaction) to adopt for design of a structure.

- Site and building period. Longer relative site to building period = higher % peak inertia.
- Stiffness of piles. Stiffer piles = higher % peak inertia
- Liquefaction trigger as a % of ULS shaking. Lower trigger = higher % peak inertia.
- Magnitude / duration of shaking. Larger magnitude = higher % peak inertia.
- Yield of structure with capacity design. Lower yield = higher % peak inertia (because the full capacity design inertia load could apply throughout the majority of the earthquake shaking).

Suggested inertia loads to consider for each phase of earthquake are presented below. The above factors can be used as guides to select a value. We typically expect that value to fall within the presented range, however, could be outside depending on the site and structure. Comparisons with the values suggested in literature (refer section 3) should be made to assist and challenge selection.

- No liquefaction: 100% peak inertia loads; no or partial liquefaction; no kinematic loads.

- After liquefaction triggering: 80% to 100% peak inertia loads; liquefaction and weakened ground; no kinematic loads.
- After liquefaction triggering: 50 to 90% peak inertia loads; liquefaction and weakened ground; cyclic displacement; consider if the crust is pushing on the piles or restraining the piles.
- Lateral spreading: 0 to 50% peak inertia loads; liquefaction and weakened ground; lateral spread displacement; consider if the crust is pushing on the piles or restraining the piles.

4 CONCLUSION

A literature review has been carried out of available guidance on liquefaction induced kinematic loads on piles. A comparison was done on the resulting pile actions for three methods. The Ashford et al (2011) p-multiplier and Sr methods had similar actions. The Cubrinovski et al (2014) method resulted in significantly lower pile actions. This method predicts greater crust loads but much smaller liquefied soil loads. A sensitivity analysis using different methods and parameters is suggested.

A literature review has also been carried out for guidance on concurrent structural inertia loads to consider with kinematic loads. There is significant variability between guidelines, and the majority of the guidelines are typically for low period structures (bridges, wharves etc). Suggestions are provided for ranges to consider for each phase of an earthquake, and factors to consider when selecting a value for design.

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