Simplified design procedures for shear piles subject to liquefaction induced lateral spread with evacuation

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ABSTRACT

The lateral load transfer between a pile and laterally spreading ground during an earthquake is a complex soil-structure interaction problem given the significant nonlinearity of the stress-strain behaviour of soils during such an event. This interaction becomes more complex when piles are used as a retaining system in close proximity to a free face/slope (e.g. river beds and coastal areas), which will likely suffer evacuation during a seismic event.

Post-liquefaction displacements due to lateral spreading can be very damaging for piles, especially in the interface between the liquefied layer(s) and the underlying stable ground. As the shear piles considered within this paper extend beyond this interface, estimating the mobilised shear forces in these reinforcing elements at depth is a key part of the design. The latter must ensure displacement compatibility between the displaced soil mass and the mobilized pile resistance.

Common practice design methods rely on an iterative procedure which combines the Newmark Sliding Block theory and Limit Equilibrium analysis to predict the permanent soil displacements and yield accelerations of the slope. This is usually followed by the use of specialized piling/retaining wall software to ensure displacement compatibility is reached. However, the guidance for the design of piles subject to lateral spread appears to not consider the detrimental effects of evacuation. This papers discusses the adaptation of this commonly accepted practice to a scenario where evacuation occurs.

1 INTODUCTION AND BACKGROUND

Liquefaction-induced lateral spread typically involves the lateral displacement of large and relatively intact blocks of soil down gentle slopes or towards a free face, such as a waterfront or riverbank. It occurs as a result of liquefaction of relatively shallow underlying strata during a seismic event and movement of non-liquefied material above the liquefied layer. It can be a major cause of damage to infrastructure located near waterfronts during and potentially after earthquakes. These displacements are usually permanent and range from a few centimeters to a few meters.

In New Zealand, the risk of liquefaction and lateral spreading was highlighted in 2010 and 2011 when large earthquakes in the Canterbury region caused significant damage to tens of thousands of houses and associated underground services, especially alongside the Avon River (Bowen et al. 2012).

Auckland Transport have decided to upgrade the resilience of Auckland City's waterfront, as part of the Auckland Downtown Infrastructure Development. This includes an existing seawall, which provides support to the Quay Street carriageway, footpaths, as well as a large number of buried utility services and provides access to the Ferry Building and Auckland's port. Some of the retained and underlying materials at this location are expected to be susceptible to liquefaction, which will likely result in lateral spreading, following significant earthquakes.

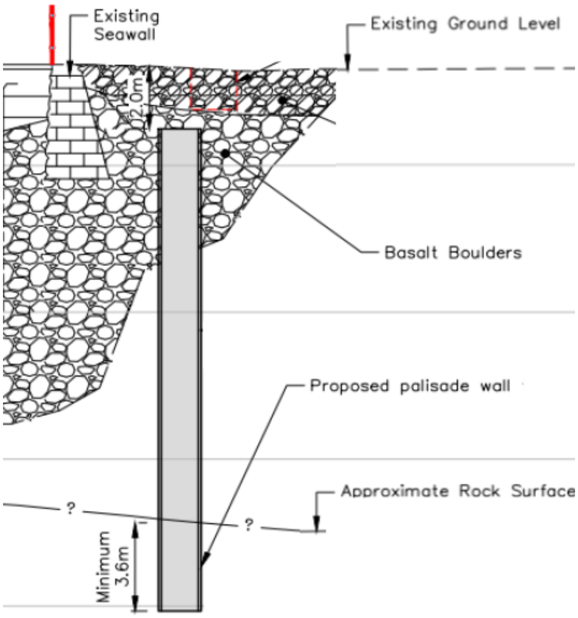


Figure 1: Typical cross section of the proposed palisade wall

To reduce lateral spread induced damage, a reinforced concrete palisade wall is proposed along a circa 300m section of the Quay Street seawall which is to be built landward of the existing seawall (Figure 1), due to infrastructure and environmental constraints, as well as the historic/heritage value of the latter. Nevertheless, a key consideration in the design was that the existing seawall would be allowed to potentially fail, during the design earthquake event, leaving the proposed palisade wall to act as a retaining solution to limit the horizontal displacements at the surface.

However, as it is often not practical or economic to design retaining walls to resist the peak ground acceleration, a commonly accepted design approach is to accept some permanent outward movement of the retaining wall. This results in a design that targets a resistance level less than the peak ground acceleration (Wood 2008). This permanent lateral displacement can be estimated using the Newmark Sliding Block method, for which there are numerous approaches given the complexities associated with the dynamic response of the retained soil behind the wall.

2 QUANTIFICATION OF LATERAL SOIL MOVEMENT

2.1 Lateral Spread

In accordance with the Newmark Sliding Block theory, a rigid block of soil is assumed to fail in a rigid-plastic manner when the ground acceleration exceeds the critical or yield acceleration of the slope. Once movement commences, it is assumed that the rigid mass will continue to slide under the earthquake inertia forces, until either the seismic acceleration reduces below the yield value or it reverses in direction. The method has been used extensively to predict the permanent movements of slopes and walls subjected to strong earthquake shaking, and can be applied to cases where the sliding movement is assumed to accumulate in only one direction, for example a downslope retaining wall.

New Zealand design guidelines (NZTA 2016) recognise the validity of this approach and recommend that "at least three different commonly accepted methods for the assessment of the displacement shall be used and the range of predicted displacements (rather than a single value) should be used in the design process". This approach was followed to estimate the likely soil block movement around Quay Street and, for the range of soil displacements/yield accelerations, the geometry of the soil block likely to displace during an earthquake event was determined by means of limit equilibrium (LE) pseudo-static analyses.

2.2 Depth of Evacuation

The assessment of the evacuation profile in front the proposed piled wall is a crucial component of the design process, as it influences the available passive resistance. If the anticipated depth of evacuation is modest, a cantilevered solution may be engineered, provided that it meets the maximum displacement criteria, otherwise additional horizontal support may be required.

Different analyses can be carried out to assist in the determination of the depth of evacuation, ranging from LE pseudo-static models to more complex finite element (FE) time history analyses. The former can be undertaken to establish all slip surfaces with a factor of safety lower than unity in front of the proposed piled wall. The interpretation of these results will suggest what portion of slope in front of the piled wall is likely to fail under the design seismic event, and therefore will help inform the likely post-evacuation profile.

Depending on the importance level of the structure to design, as well as the extent of the likely evacuation, the post-earthquake profile may require confirmation by means of FE dynamic analyses, using a range of inferred ground motion records. This procedure is outside of the scope of this paper but can be found in Neves et al (2020).

3 PROPOSED SIMPLIFIED DESIGN PROCEDURE

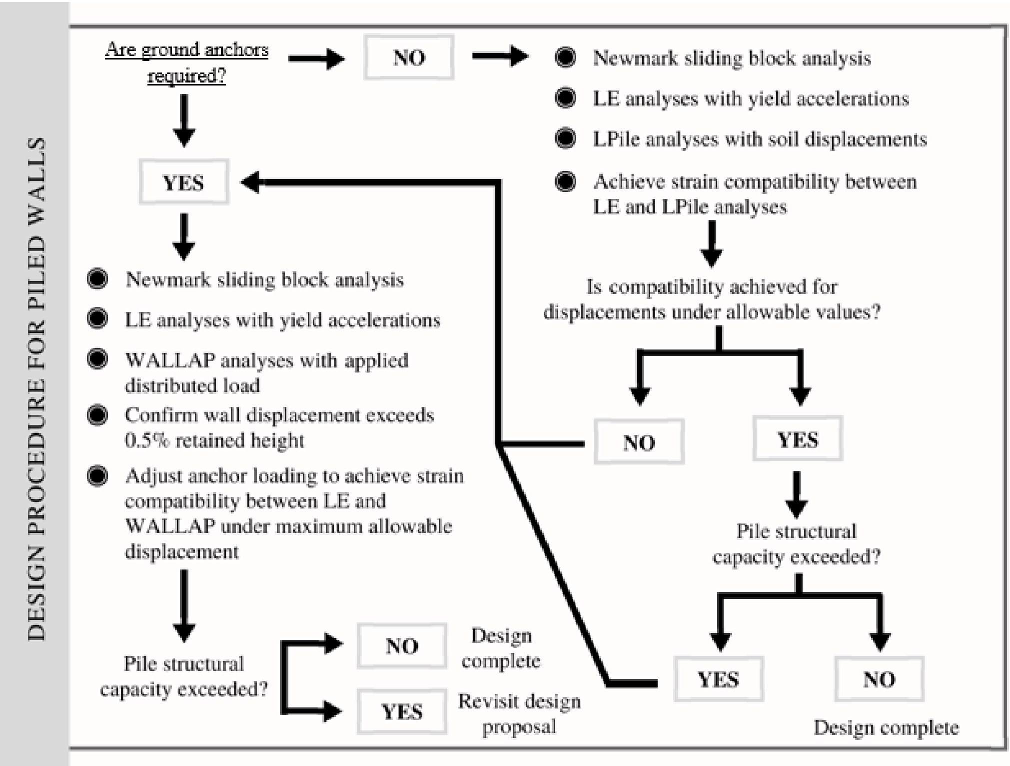


Figure 2: Proposed design flowchart

The following two-part methodology has been developed for the design of piled walls subject to liquefaction lateral spread with evacuation. It combines two different approaches to this problem, the Displacement Based Design and the Equivalent Shear Force Method. Each of these approaches respectively applies to either an unrestrained or restrained ground displacement design case and makes use of readily available commercial software.

This design procedure initially assumes that a cantilevered piled wall may suffice to resist the earthquake loading. Failing to reach an acceptable outcome, the initial assumptions are revisited by considering some form of lateral restraint at the top of the proposed retaining wall (i.e. tieback or anchor). Nevertheless, if from the offset, the cantilevered option is not considered as a feasible alternative, then the designer may choose to ignore the first part of this design procedure. The flowchart on Figure 2 summarises the proposed design procedure and is described in the following section.

3.1 Design Methodology for Cantilevered Piled Walls (Displacement Based Design)

The following step-by-step methodology is proposed for the design of cantilevered piled walls subject to liquefaction lateral spread with evacuation:

1. Complete a Newmark Sliding Block analysis to estimate the free field permanent soil displacements (i.e. ignoring the presence of the piled wall) for a given range of horizontal yield accelerations;
2. Apply those yield accelerations in a LE slope stability software to determine the minimum required shear force on the piles to achieve a code compliant Factor of Safety (FoS). Plot the results on a soil displacement versus pile shear force plot;
3. Apply the soil displacements from 1) in LPILE (or equivalent) and determine the mobilised shear forces on the pile [Fig. 3 (a)]. Plot results on the soil displacement versus pile shear force plot from 2), Figure 3 (b);

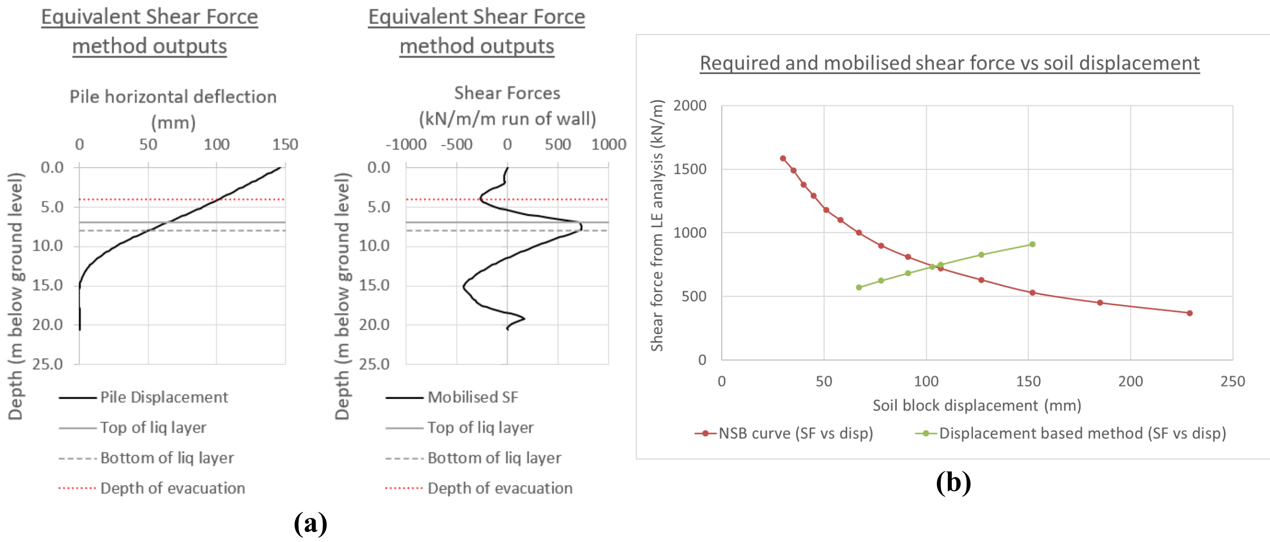


Figure 3: Displacement Based Design outputs: (a) Example of NSB convergence results and (b) soil displacement, pile deflection and mobilised shear forces for displacement compatibility

4. Compare the mobilised shear forces in 3) with the required shear forces in 2) until both curves on the soil displacement versus pile shear force plot intersect (convergence is achieved – Figure 3 (b)); and
5. Compare bending moments and shear forces on the piles against pile capacity and pile displacement against allowable values.

When convergence at the end of the procedure listed above occurs for displacements beyond the acceptable limits or if the piles are loaded beyond their capacity, some form of lateral restraint may have to be included (i.e. tieback or anchor) by following the methodology presented in the following section.

3.1.1 Existing Modelling of Lateral Soil Movement on LPILE (or equivalent)

LPILE allows for a soil movement to be applied to a pile in order to model its soil-pile interaction. The software incorporates the free-field soil displacements as movements on the "free" ends of the soil springs, which inherently assumes that the soil deforms as a shear beam. For the purpose of lateral spreading, it is commonly assumed that the displacement profile should be applied as shown in Figure 4 (a) (MBIE, 2016). This assumes that full lateral displacement takes place above the liquefied layer, decreasing linearly to zero throughout its depth. In the cases where more than one layer at depth has been identified as liquefiable, a case by case assumption regarding the soil lateral displacement profile with depth should be made. At Quay Street, it has been assumed that full lateral displacement will occur above the top of the deepest liquefiable layer.

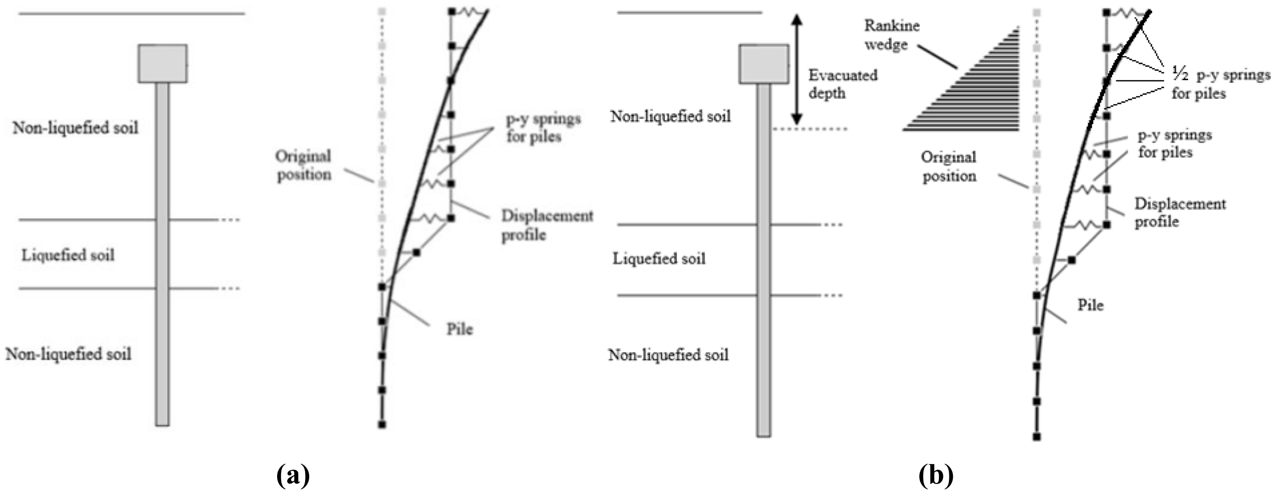


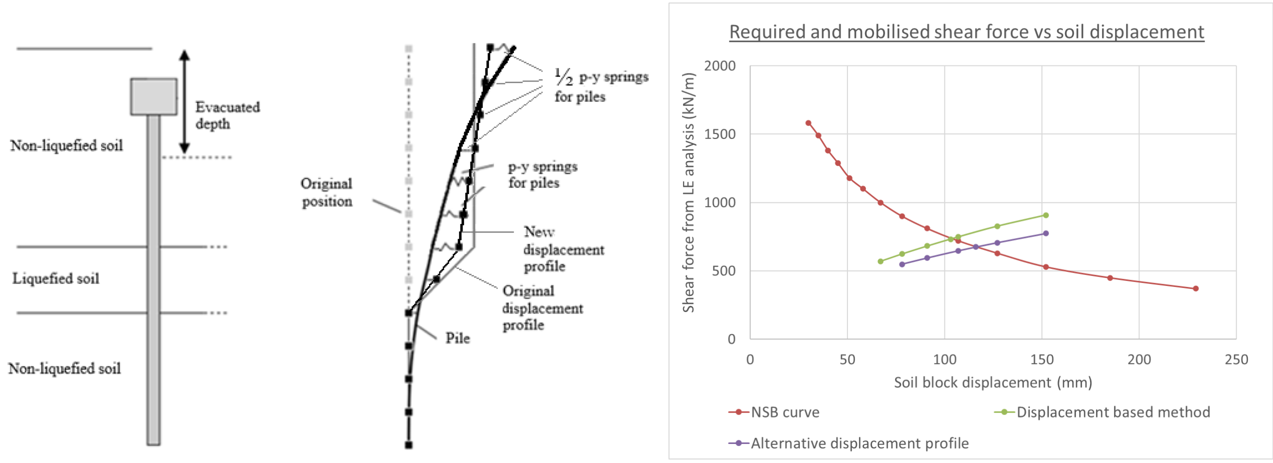
Figure 4: (a) Applied displacement profile for lateral spread analysis and (b) LPILE lateral spread modelling with evacuation, adapted from MBIE (2016)

One of the limitations of LPILE, is the fact that the software cannot accommodate for different ground surface levels in front and behind the pile. Therefore, and in order to capture the loss of passive support in front of the pile, the depth of evacuation needs to be somehow incorporated in the analysis. The proposal outlined in this paper takes that into account by softening the soil springs above the depth of evacutation (Fig. 4 b).

Additionally, a Rankine passive wedge load is proposed, acting on the active side of the piled wall, as recommended by the California Department of Transportation (Caltrans 2013), highlighted in the Transportation Japanese Highway Specification (Arduino et al 2017 and JRA 2002) and corroborated by centrifuge testing (Brandenberg et at 2004). These references recommend the design of piles against bending failure to account for the non-liquefied crust exerting passive earth pressure on the pile. This assumption is considered conservative as it ignores the rotational restraint provided by the non-liquefied crust.

3.1.2 Alternative Lateral Soil Block Movement

The procedure described in Section 3.1.1 is deemed as a conservative approach to model the effects of lateral spreading on piles subject to evacuation. However, for this project in particular, the method was further refined, based on an FE time history analysis validation procedure. In this particular scenario, it was observed that a linear variation displacement profile, as opposed to a uniform soil block movement, better approached the results of the FE analysis, and negated the need to apply the additional Rankine passive wedge (Fig. 5 a), contrary to the findings of Brandenberg et at (2004). However, it should be noted that the centrifuge testing undertakin in the latter did not effectively account for a scenario where soil evacuation took place. Moreover, the new displacememnt profile applies a larger displacement to the top part of the piles, where soil springs have been halved, which in turn result greater structural stresses within the pile.



(a) **(b)**

Figure 5: (a) Final LPILE lateral spread modelling with evacuation applied at Quay St and (b) NSB convergence results

The alternative lateral soil movement profile, although varying linearly with depth, assumes the same average value throughout the depth of lateral spreading as the original proposal. The average value is still used in the Newmark Sliding Block analysis to estimate the permanent soil displacements for a given range of horizontal yield accelerations, thus resulting in the same shear force demand curve. Figure 5 (b) illustrates the differences between the convergence procedure for both approaches.

It is noted that in order to be able to adopt this alternative approach, the gradient of the imposed soil block displacement should be calibrated with further analysis, as exemplified in Neves et al (2020). This alternative approach has resulted in a significant reduction of the maximum shear forces and bending moments experienced by the piles (maximum structural capacity) for similar soil block displacements and shear forces at the slip plane.

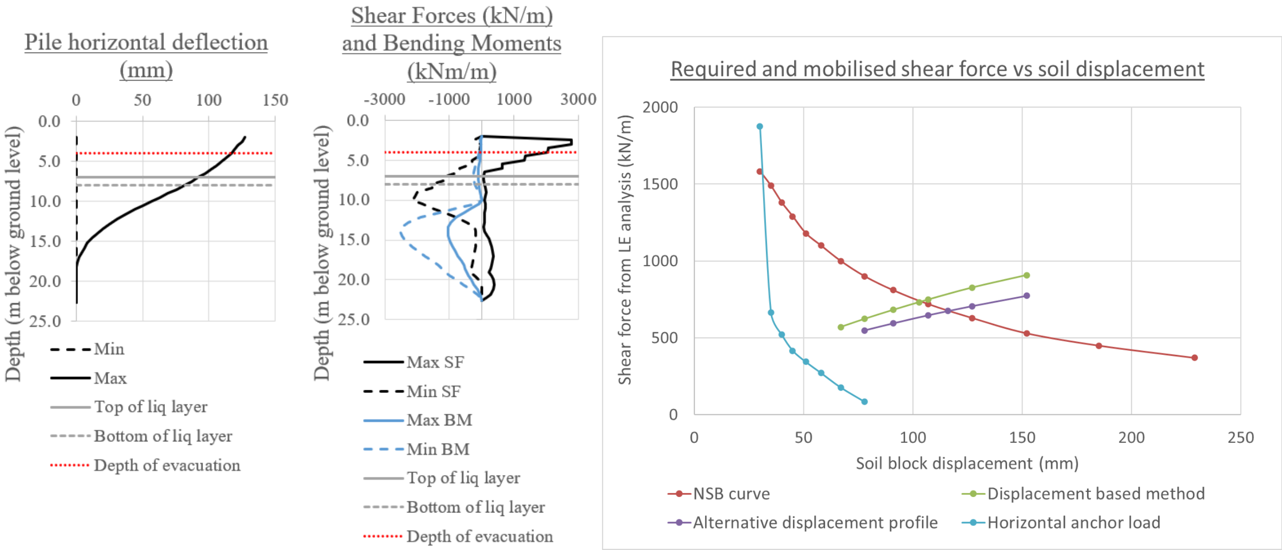
3.2 Design Methodology for Tied-Back/Anchored Piled Walls (Equivalent Shear Force Method)

By impeding the free movement of the piled wall, the tied-back/anchored pile approach to the problem differs from the previous scenario. Nevertheless, and since a piled wall is still unlikely to be a stiff retaining wall, a reduction in the horizontal seismic acceleration may still be taken into account, provided that the wall deforms more than 0.5% of the retained height (Wood and Elms 1990).

In this case, the following steps are proposed:

1. Complete a Newmark sliding block analysis to estimate the permanent soil displacements for a given range of horizontal yield accelerations;
2. Apply those yield accelerations on a LE software to determine the minimum required shear force on the piles to achieve a code compliant FoS. Plot the result on a soil displacement versus pile shear force plot;
3. Select allowable displacement for the piled wall, confirming it exceeds 0.5% of the retained height;
4. Distribute the shear force associated with the displacement selected in 3) as point forces between the top of the pile and the depth of the slip plane on a retaining wall design software (e.g. WALLAP) and also model the proposed tieback/anchor;
5. Adjust anchor load until pile displacement matches the selected soil block movement and confirm the maximum allowable elongation of the tieback/anchor is not exceeded;
6. Compare bending moments and shear forces on the piles against pile capacity (Figure 6 a); and

7. If anchor capacity, anchor elongation and/or pile stresses exceed their limits, allow for further wall displacement or adjust pile size and spacing and re-do steps 3 to 5 (Figure 6 b);.



(a) **(b)**

Figure 6: Equivalent Shear Force Method outputs: (a) NSB convergence and (b) soil displacement, pile deflection and mobilised shear forces for displacement compatibility

3.2.1 Modelling of Evacuation and Seismic Conditions on WALLAP (or equivalent)

Contrary to LPILE, modelling on WALLAP allows for a given construction sequence to be considered in the analysis. The Winkler spring-based analysis allows wall deflections and internal forces to be computed, as well as the inclusion of external forces acting on the wall. However, the adjacent ground movements away from the wall cannot be predicted.

In scenarios where evacuation is anticipated, the following general staging is proposed:

a. Step-up the initial problem geometry and pile with elastic properties. Liquefiable materials should be modelled with their full strength parameters at this initial stage;
b. Install tie-back/anchor with prestess (if applicable);
c. Change flexural stiffness of piled wall to account for concrete cracking;
d. Change soil properties of liquefiable materials;
e. Change problem geometry to reflect evacuation scenario;
f. Apply point loads at appropriate spacings, equivalent to the required shear force from the LE models, between the slip plane and the top of the pile.

4 CONCLUSION

It is generally accepted that the modelling of the lateral load transfer mechanism between pile and soil, in the case of a lateral soil displacement problem (e.g. lateral spread), is best approximated by specialist pile analysis software that can capture the non-linearity of this interaction. Nevertheless, no guidance appears to be available on the procedure to follow when dealing with lateral soil movement, combined with an evacuation scenario. This paper discusses a possible methodology to adopt under these circumstances, which makes use of readily available software and ensures displacement compatibility between the required shear forces to ensure overall stability and the predicted soil/pile displacements. However, two distinct approaches are proposed, one for a cantilevered piled wall and a second methodology for a tied-back/anchored wall.

However, it is noted that some uncertainty still remains in the choice of shear force to use on the strain compatibility procedure. The conservative view is that it should correspond to the value on the shear force diagram at the slip plane depth, corresponding to the integral of the mobilised soil reaction from the top of the pile until this depth. In other words, corresponding to the sum of the forces that the soil applies to the pile up to that depth. A different interpretation is that one should consider the change in shear that occurs below the slip plane for this procedure, thus comparing the load transfer mechanism to a beam with an applied point load, and which results in a marked change in the shear diagram.

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