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Mitigation of liquefaction induced lateral spread using jet grout shear walls

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ABSTRACT

The load transfer mechanism between ground subject to liquefaction induced lateral spreading and jet grout shear walls is complex. The shear walls comprise rows of jet grout columns, constructed in a primary-secondary sequence to ensure a minimum overlap, and are aligned parallel to the direction of the lateral movement.

The displacements experienced by the shear walls generate shear, compressive and tensile stresses that need to be accommodated by the treated soil block without exceeding its capacity. This is particularly relevant in the overlapping sections of the shear walls, to ensure that all columns act as a single block, as well as along the external faces of the block, where the maximum compressive and tensile forces are generated.

Given the limited tensile and shear capacity of the jet grout columns, correctly estimating the mobilised shear forces in the interface between columns, as well as compressive and tensile forces within the toe and/or heel of the reinforced soil block are crucial. To ensure an appropriate load transfer throughout the height of the shear wall, common practice design methods rely on empirical column overlap versus diameter ratios to establish the minimum geometry requirements of the shear walls.

The available guidance for the design of in situ soil treatment shear walls appears to be limited to static load cases, predominantly underneath the side slopes of embankments to enhance slope stability. This paper discusses the adaptation of this commonly accepted practice to a scenario where jet grout shear walls resist liquefaction induced lateral spread.

1 INTRODUCTION AND BACKGROUND

Liquefaction-induced lateral spread typically involves the lateral displacement of large and relatively intact blocks of soil at shallow grade 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 importance of liquefaction and lateral spreading was highlighted in 2010 and 2011 when 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).

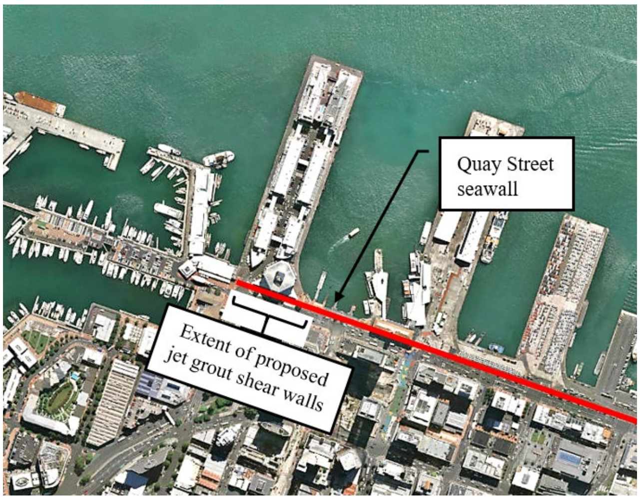
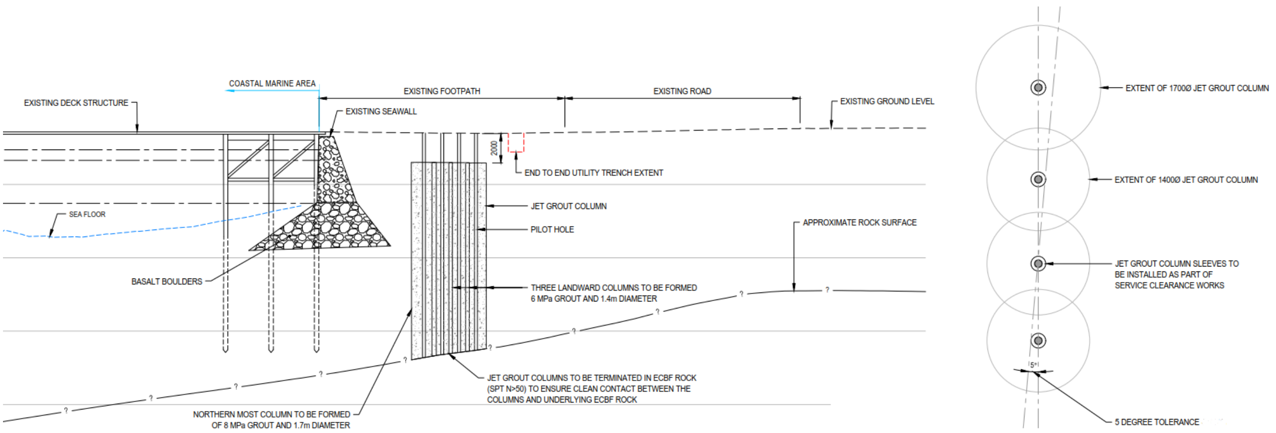


Figure 1: Site location plan [OpenStreetMap, 2018]

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, approximately 600m long, which provides support to the Quay Street carriageway and footpaths, as well as a large number of buried utility services and access to the Ferry Building and Auckland's port (Fig. 1). 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.

To prevent lateral spread, a series of jet grout shear walls is proposed along a circa 110m section of the Quay Street seawall which is to be built landward of the existing seawall [Fig. 2 (a)], due to existing infrastructure and environmental constraints, as well as the historic/heritage value of the existing seawall.



(a) (b)

Figure 2: Typical cross section (a) and plan (b) of the proposed jet grout shear walls

The proposed design comprises rows of four overlapping jet grout columns to form a shear wall of improved ground. The columns are constructed in a primary-secondary sequence to ensure a minimum overlap between adjacent columns. Each proposed shear wall comprises a 1.7 m diameter seaward column followed by three 1.4 m diameter landward columns [Fig. 2 (b)]. The columns are constructed at a maximum centre-to-centre spacing of 1.2 m in the north south direction, with each row at 3 m centre-to-centre spacing in the east west direction. All columns are terminated within the underlying East Coast Bays Formation (ECBF) rock (Standard Penetration Test (SPT) N value greater than 50), which underlies the liquefiable superficial soils.

2 DESIGN APPROACH FOR STATIC LOADING

The available guidance on the design of in situ soil treated shear walls primarily focuses on the use of these elements to support embankments and/or prevent stability failure of their side slopes (Federal Highway Administration (FHWA), 2013). It is predominantly based on the concept of Area Replacement Ratio (ARR) and on typical ratios between column overlap (e) and column diameter (d).

The overall design approach, as proposed by FHWA (2013) follows the following steps:

1. Define the geometry of the columns (diameter and minimum overlap). The minimum overlap should consider the maximum allowable column verticality deviation;
2. Decide on the minimum unconfined compressive strength (UCS) and assess the strength and deformability parameters of the jet grout columns;
3. Select a trial ARR for the treated soil block (i.e. definition of the maximum spacing between shear walls);
4. Undertake a global/overall stability check;
5. Undertake an external stability check (overturning, sliding and toe crushing);
6. Undertake an internal stability check (vertical shear between columns and extrusion between shear walls);
7. Confirm if design is code complaint and meets design requirements, if not re-start the process adjusting geometry and/or strength inputs.

The same document (FHWA, 2013) offers guidance into the preliminary sizing and spacing of the shear walls and offers guidance on the equations to use to complete the checks in points 5 and 6. For the global stability check, limit equilibrium (LE) slope stability software can be used determine the Factor of Safety (FoS) of the proposed design.

3 PROPOSED DESIGN APPROACH FOR SEISMIC LOADING

The limitation of the procedure listed in Section 2 is that it does not explicitly account for dynamic/seismic loading as a load case acting on the proposed shear walls. An adaption of the design approach proposed by FHWA (2013) is outlined below, to account for a scenario where jet grout shear walls resist liquefaction induced lateral spread.

The adapted design approach is as follows:

1. Define the geometry of the columns (diameter and minimum overlap). The minimum overlap should have in consideration the maximum allowable construction deviation;
2. Decide on the minimum unconfined compressive strength (UCS) and assess the strength and deformability parameters of the jet grout columns;
3. Select a trial ARR for the treated soil block (i.e. definition of the maximum spacing between shear walls);
4. Undertake a global/overall stability check
5. Undertake a Newmark Sliding Block analysis to estimate design horizontal yield accelerations for the assumed range of permanent displacement (refer to Section 3.1);
6. Assess the total active and passive seismic forces acting on the reinforced block (refer to Section 3.2);
7. Undertake an external stability check (overturning, sliding and toe crushing);
8. Undertake an internal stability check (vertical shear between columns and extrusion between shear walls);

9. Confirm if design is code complaint and meets design requirements, if not re-start the process adjusting geometry and/or strength inputs.

As with any other form of (embedded) retaining element, it is often not practical or economic to design them to resist the peak ground accelerations, especially in regions of high seismicity. A commonly accepted design approach is to accept some permanent outward movement of the retaining wall, which results in the design targeting a resistance level that is less than the peak ground acceleration [Wood 2008].

In the case of jet grout shear walls, and despite the fact that the proposed shear walls act as a "rigid" block, it is anticipated that some permanent displacement will occur due to rotation and/or translation of the block, and therefore result in a reduction in the seismic demand acting on the treated block.

To confirm the likely range of permanent displacements FE dynamic analysis can be undertaken, 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). For preliminary purposes, and/or if the project importance does not merit the undertaking of this sort of detailed analysis, conservative displacements between 10 and 20 mm are recommended based on the findings from the Auckland Downtown Infrastructure Development project and comparison with other retaining wall types embedded within similar ground conditions.

The associated range of yield accelerations to these permanent lateral displacements can be estimated using the Newmark Sliding Block method (Fig. 3), for which there are numerous approaches given the complexities associated with the dynamic response of the retained soil behind the wall.

3.1 Quantification of lateral soil movement

In accordance with the Newmark Sliding Block theory, a sliding 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 sliding 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 by many practitioners and researches 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.

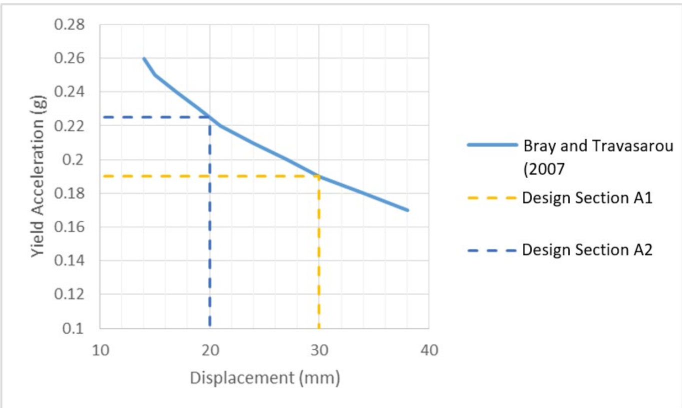
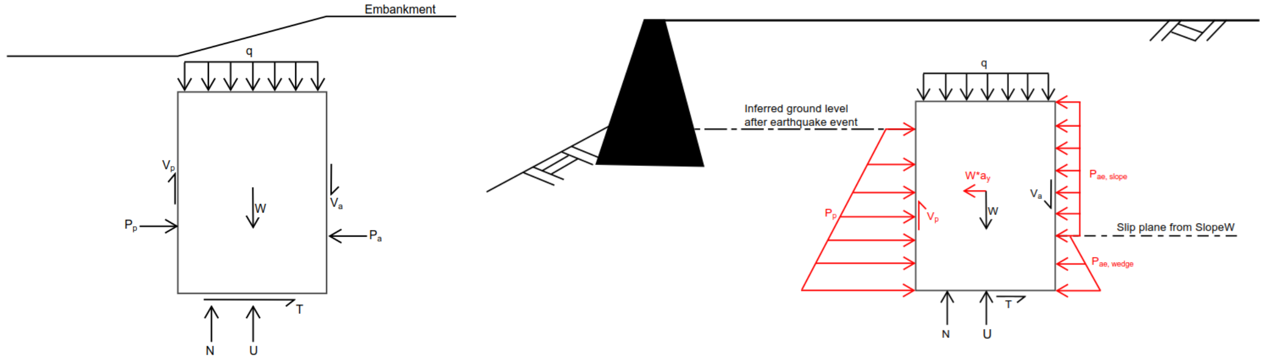


Figure 3: Example of range of accelerations for design sections A1 and A2 based on Bray and Travasarou (2007)

New Zealand design guidelines (NZTA 2016) recognise the validity of this approach and recommend "at least three different commonly accepted methods for the assessment of the displacement 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 yield accelerations associated with the selected range of soil block permanent displacements for a range of design sections (Fig. 3). These yield accelerations were then used in the LE pseudo-static analyses.

3.2 Quantification of lateral spreading load

In the design scenario covered by this paper, the stability of the jet grout block was assessed using the free body diagram depicted in Figure 4, adapted from FHWA (2013). As part of the down slope soil mass is anticipated to evacuate during a seismic event, a reduction in the passive forces acting on the shear has been considered in the pseudo-static analysis. This has been derived based on an assessment of the post-seismic soil profile in front of the shear wall.



(a) (b)

Figure 4: Free body diagram of forces acting on shear wall for (a) static analysis and (b) a seismic event, adapted from FHWA (2013).

Where:

- q is the soil surcharge;
- $P_{ae,\ slope}$ is the lateral spreading load;
- $P_{ae,\ wedge}$ is the seismic active wedge pressure, based on Mononobe-Okabe (M-O) theory and Wood and Elms (1990);
- P_p is the passive pressure based on M-O theory, albeit adjusted for the post-seismic soil profile;
- V_a is the shear on the active side of the shear wall;
- V_p is the shear on the passive side of the shear wall, adjusted for the post-seismic soil profile;
- a_y is the yield acceleration as determined in Section 3.1;
- W is the self-weight of the shear wall;
- W^*a_y is the shear wall inertia;
- T is the base shear at the base of the shear wall;
- U is the buoyancy force acting on the base of the shear wall; and
- N is the normal force from the founding soil acting on the base of the shear wall.

The quantification of the active seismic load applied to the block has been divided into two components. Given the uncertainties in the quantification of the magnitude of the lateral spreading load ($P_{ae,\ slope}$) acting on the shear walls, the LE modelling has been used to provide an indicative estimate. Within the LE analysis a pile reinforcement has been modelled, at the landward face of the jet grout shear wall (Fig. 5), to determine the required shear force to achieve a unitary FoS. Note that this procedure has considered a yield acceleration as indicated in Section 3.1. Having assessed the magnitude of the slip surface driving force, it has been assumed that this load is uniformly distributed between the top of the shear wall and the slip plane.

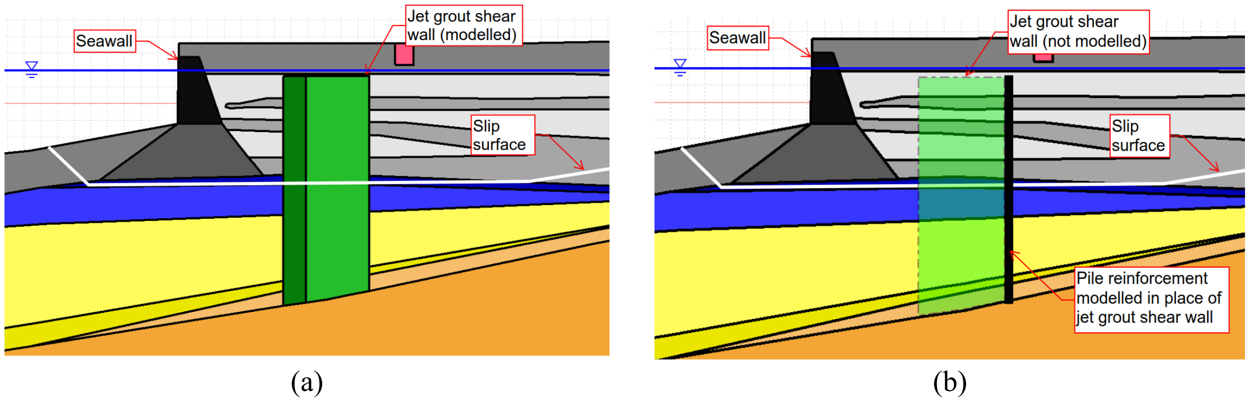


Figure 5: (a) LE model used to assess overall stability and (b) LE model with pile reinforcement to estimate lateral spreading load.

A traditional M-O approach has been consider to derive the active pressures wedge ($P_{ae,\ wedge}$) acting on the shear wall below the slip surface plane.

3.3 External stability checks

The external stability checks, which include overturning, sliding and bearing capacity failures, have been undertaken using the methodology proposed in the FHWA design manual (FHWA, 2013). The only significant adaptation has been the use of the alternative forces acting on the shear wall as illustrated in Figure 4 (b).

3.4 Internal stability checks

Similarly to the external stability checks, the procedures listed in the FHWA design manual (FHWA, 2013) have been followed to complete the internal stability checks, which include internal slip, shearing on vertical planes and outside toe crushing failures.

Given the nature of these shear walls (formed by overlapping jet grout columns), a concentration of shear forces tends to occur at the planes of overlap due to the reduction in cross sectional area. As a result, a minimum chord length between adjacent columns is required to provide adequate shear capacity at these locations and ensure the columns act as a rigid block. FHWA (2013) offers guidance on the typical ratio of e/d to adopt, with "e" and "d" being the column overlap and diameter, respectively. However, these have been found, for the scenario in analysis, to exceed the minimum requirements by over 50%, which has been corroborated by means of using finite element (FE) analysis (Neves et al. 2020). It is worth noting that the adoption of a "leaner" spacing between columns will inevitably lead to stringent verticality controls and acceptance criteria during construction.

However, it has been found that the check against the outside toe crushing failure mechanism using the procedure within the FHWA (2013) design manual does not appear to accurately capture the state of stress at this location during a seismic event.

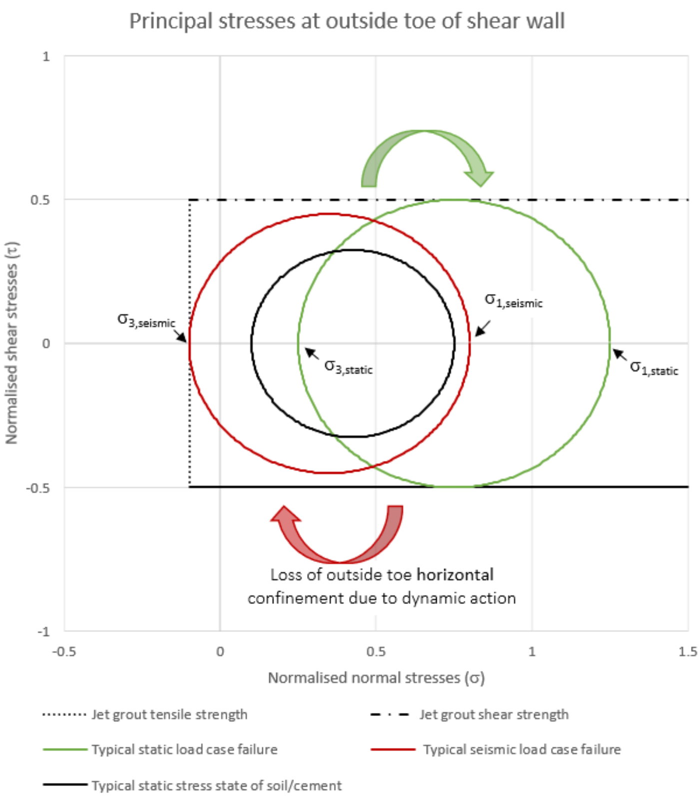


Figure 6: Principal tensions at failure of the outside toe for static and seismic load conditions.

For a static load case, the capacity against crushing of the toe is dependent not only on the inherent compressive strength of the jet grout material, but also on the favourable effect of the passive lateral earth pressure at that point. From a principal stresses perspective, and as illustrated in Figure 6, this lateral pressure ($\sigma_3 > 0$) increases the maximum allowable principal stress (σ_1) that the material can reach.

However, and as identified in the FE analysis (Neves et al. 2020), the cyclic movement that occurs during a seismic event can momentarily result in a near complete loss of soil horizontal confinement at the outside toe. In turn, this results in an unconfined uniaxial state of stress, where values of σ_3 are progressively reduced until they reach the jet grout tensile strength capacity. As such, the outside toe crushing failure mechanism has actually been found to be predominantly governed by the tensile strength of the grout, rather than its compressive strength.

Nevertheless, the decrease in σ_3 also results in a decrease of the maximum allowable σ_1. Therefore, it is proposed that for this simplified method, the capacity against crushing of the outside toe (q_{all}) is assessed in two parts:

1. Compare the maximum compressive stress at the outside toe with only the compressive capacity of the jet grout material, i.e. ignoring any confining effect from the soil in front of the wall. Equation 65 on the FHWA (2013) should then be adapted as follows:

$$\sigma_1 \leq q_{all} = \frac{2s_{dm}f_v}{F_c}$$

2. Compare the maximum tensile forces at the outside toe with the tensile capacity of the jet grout material. To do so the following equation is proposed:

$$\sigma_3 \approx \sigma_1 - 2 \times s_{dm} \leq \frac{f_t}{F_{tt}}$$

Where:

- s_{dm} is the shear strength of the jet grout material;
- f_v is the coefficient of variation;
- F_c is the FoS against crushing of the toe;
- F_{tt} is the FoS for the allowable tensile strength at the toe;
- σ_1 is the maximum principal stress (maximum compression at outside toe);
- σ_3 is the minimum principal stress (maximum tension at outside toe); and
- f_t is the allowable tensile strength of the jet grout material.

Given the difficulty in accurately predicting the maximum tensile forces generated at the toe of the shear wall, and in the absence of a more accurate analysis, a conservative value of F_{tt} should be adopted to ensure that crushing of the outside toe is not the governing failure mechanism of the shear wall.

4 CONCLUSION

The use of soil-cement columns to form embedded shear walls is relatively common. These are often adopted to support road/rail embankments, designed to withstand static lateral loading and enhance slope stability, with readily available design guidance. However, no case histories having been found in our research of scenarios where overlapping jet grout columns shear walls have been explicitly designed to resist liquefaction induced lateral spread with evacuation.

This paper presents an adaptation of the design guidance for jet grout shear wall as provided by the FHWA (FHWA, 2013) to account for seismic and lateral spread loading. The proposed methodology also makes use of the Newmark Sliding Block method to reduce the seismic demand (yield acceleration), based on an assumed range of permanent displacements. The latter have been based on engineering judgement and past experience, and corroborated by FE analysis (Neves et al. 2020). A proposal is also included to estimate the lateral spread loading acting on the shear walls.

External stability checks are proposed to be carried out using the methodology presented in the FHWA design manual (FHWA, 2013), with the exception of the forces acting on the shear wall and their quantification. With regards to the internal stability check, an adaption of the FHWA guidance is proposed in relation to the check against outside toe crushing failure. This modification accounts for the altered state of stress at this location during a seismic event, which has been corroborated by FE modelling (Neves et al. 2020).

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