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Horizontal Soil Mixed Beam Ground Improvement as a Liquefaction Mitigation Method Beneath Existing Houses

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ABSTRACT

Ishihara (1985) recognised that a thick non-liquefying crust overlying liquefying soils would reduce the consequences of liquefaction (i.e., sand boils, loss of bearing capacity and differential settlement). In Christchurch, in the aftermath of the 2010-2011 Canterbury Earthquake Sequence (CES), detailed engineering assessments of nearly 60,000 single-family houses combined with a comprehensive regional scale geotechnical investigation, clearly showed that less structural damage occurred in liquefaction-prone areas containing an intact, relatively stiff non-liquefying crust with minimum thickness of approximately 3 m. To increase the resilience of the post-CES existing Christchurch residential housing portfolio (which has been repaired) to future liquefaction damage, the use of shallow (i.e., ≤ 4 m deep) ground improvements to construct stiff, non-liquefying crusts to mitigate the consequences of liquefaction of the underlying soil layers was evaluated. This paper presents the results from the in-situ vibroseis dynamic (T-Rex) load testing and dynamic numerical simulation to examine the liquefaction triggering of Horizontal Soil Mixed (HSM) beam ground improved soils compared to natural soils. The shake testing of the HSM beam ground improvement panels as well as the numerical simulations demonstrated that HSM ground improvement resulted in reduction in the maximum cyclic shear strain (γ) and excess pore water pressure (r_u) induced in the improved soil and hence provide an improvement in liquefaction resistance.

Introduction

The 2010 – 2011 Canterbury Earthquake Sequence (CES) caused widespread liquefaction-related land and building damage (described in Rogers et al., 2015), affecting 51,000 residential properties in Christchurch, including 15,000 residential houses damaged beyond economical repair. In addition, as a result of the ground surface subsidence caused by the CES, the liquefaction vulnerability has increased in some parts of Christchurch (Russell et al. 2015), increasing the vulnerability of the repairable houses to liquefaction-related damage in future earthquake events. In the suburbs most vulnerable to liquefaction damage, the CES revealed the importance of constructing robust, stiffened foundations capable of resisting the damaging effects of liquefaction (i.e. angular distortion, lateral stretch and loss of ground support) or instead undertaking ground improvement to mitigate the damage caused by liquefaction. In response, the New Zealand Earthquake Commission (EQC) funded an extensive shallow ground improvement trial program to evaluate the efficacy of various shallow ground improvement methods for reducing the liquefaction vulnerability of the residential housing portfolio.

There are a number of shallow ground improvement methods suitable for installation on cleared residential properties (i.e., the houses that were damaged by the CES beyond economic repair and will be rebuilt). However, options for improving sites where there are existing buildings

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which are economic to repair (which do not involve removing or demolishing the building) are limited. A new ground improvement approach, comprising the construction of Horizontal Soil Mixed (HSM) beams beneath existing houses was developed. The method involves mixing of injected grout into in-situ soils to construct a series of discreet HSM beams. The development of the construction methodology is discussed in detail in Hunter et al. (2015).

The main purpose of the HSM ground improvement method is to increase the thickness of the non-liquefying crust beneath existing houses. To achieve this, the HSM beams are installed within the shallowest layer of soil that would otherwise be vulnerable to liquefaction (the target soil layer). While it would be possible to construct the HSM beams to form a continuous soil cement raft (i.e., with the HSM beams overlapping each other so that there is no unmixed soil between the beams), this was not considered economic. The HSM beams were therefore designed to prevent liquefaction, at the design ground motions, between the HSM beams within the target soil layer by acting to confine the interstitial soil that remains between the beams.

Accordingly, the effectiveness of the beams at confining the interstitial soil and preventing the triggering of liquefaction within it is a key design question for a given HSM beam layout. In order to examine this, studies of a typical layout of HSM beams have been undertaken. These studies comprised in-situ vibroseis dynamic (T-Rex) load testing and dynamic numerical simulation.

T-Rex Shake Testing of the HSM Beam Ground Improvement

Panels of HSM beams were constructed at three sites in eastern Christchurch (Sites 3, 4 and 6, shown in Wissmann et al., 2015). The T-Rex testing undertaken for the HSM beams was part of a wider trial assessing a number of different shallow ground improvement techniques. van Ballegooy et al. (2015a and b) discusses this trial and the T-Rex shake testing methodology in greater detail. A plan view and cross section schematic of the construction of a double row of the HSM beams, the instrumentation layout, T-Rex base plate layout and shake testing direction relative to the beams for one of the panels at Site 4 is shown in Figure 1.

The vibroseis testing dynamically applied oscillating shear loads from the ground surface at both the unimproved and the HSM beam improved panels. An array of 3D geophones and pore water pressure transducers (PPTs) were installed in the ground directly below the T-Rex shaker to indirectly measure the cyclic shear strain, γ (from relative displacements between adjacent geophone sensor locations) and the excess pore water pressure (r_u). These measurements were collected over a range of applied shaking levels and the γ results for one of the double row of HSM beam panels constructed at site 4 and the double row of HSM beam panel constructed at Site 6 are shown in Figure 2 and compared to the adjacent natural soil panels. It is noted that the shaking at all the HSM beam panels was undertaken in the perpendicular direction apart from the HSM beam panel at Site 4 which was shaken in both directions (refer to Figure 1).

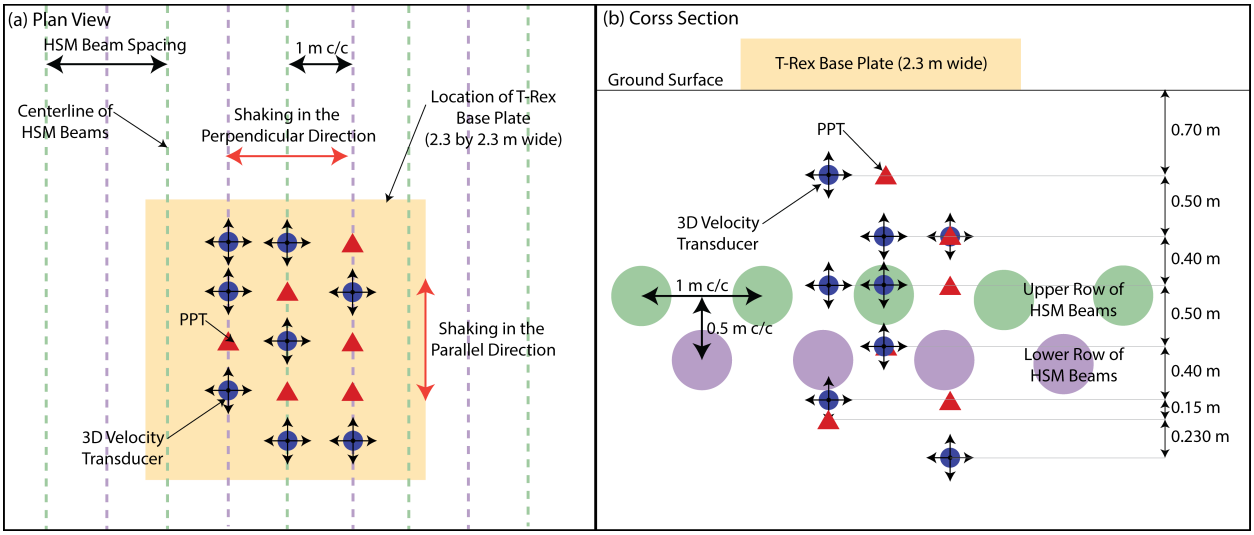

Figure 1. (a) Plan view and (b) cross section of the instrumentation setup, including the Pore water Pressure Transducers (PPT) and three dimensional (3D) geophones, relative to the T-Rex base plate and the HSM beams at the third test panel at Site 4.

Because the T-Rex shaker applies shear loads at the ground surface to a 2.3 m square plate, the γ profiles decay relatively rapidly with depth. The results shown in Figure 2 indicate that for each of the applied shear stress levels at Site 6 and the higher applied shear stress levels at Site 4 (> 15 kPa), the γ in the soils between the HSM beams were considerably reduced as a result of the beams compared to the adjacent natural soil panels, decreasing the potential for development of r_u and hence liquefaction triggering under cyclic loading. However, at the smaller applied shear stress levels at Site 4 (< 10 kPa), the γ in the soils between the HSM beams appeared to increase as a result of the beams compared to the adjacent natural soil panels. Figure 2 also shows the Cone Penetration Test (CPT) tip resistance (q_c) and soil behaviour type index (I_c) as well as the crosshole shear wave velocity (V_S) of the natural soils surrounding the HSM panels (shown as the lighter blue traces) and within the HSM panel, collected prior to the HSM beam construction (shown as the darker blue traces). At Site 6 the CPT and V_S profile at the HSM panel appears the same as the surrounding soil, whereas at the Site 4 panel location the V_S profile appears to have lower values compared to the surrounding soils, possibly indicating that the HSM beams were constructed in a local soft spot. This would increase the γ values and make the interpretation of the relative comparisons between the HSM beam improved soil results and adjacent natural soil results difficult and could possibly explain the increase in γ values at the lower applied shear stresses.

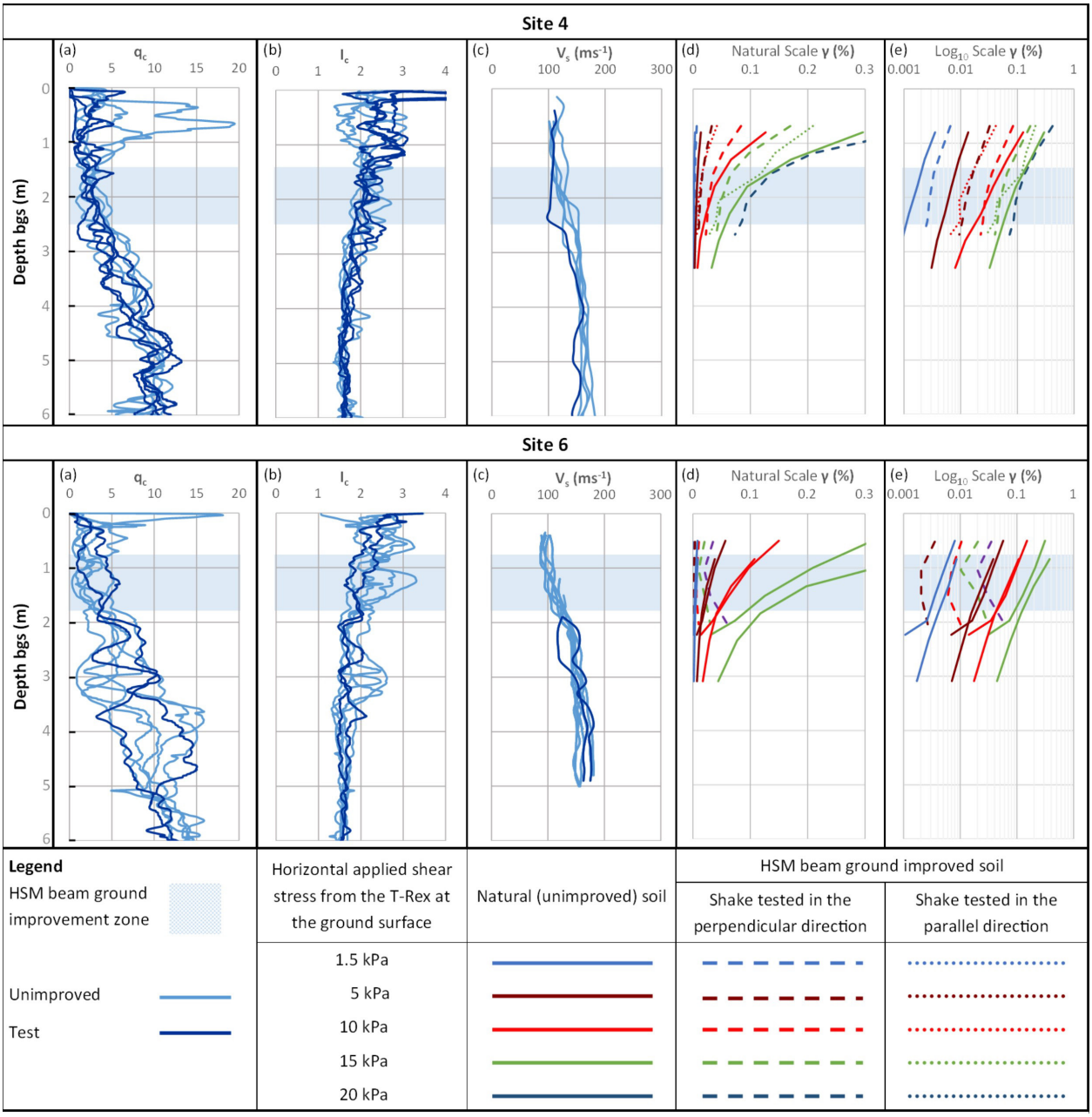

Figure 2. Natural soil CPT q_c and I_c profiles, crosshole V_S profiles and T-Rex γ profiles. It is noted that the results from the double row of HSM beams at Sites 3 and two panels at Site 4 are not shown because the HSM beams for these panels were not in a constructed correctly in the regular pattern shown on Figure 1 (van Ballegooy et al., 2015b).

Dynamic Numerical Modelling of the HSM Beam Ground Improvement

In order to further assess the effectiveness of the HSM beams in mitigating the effects of liquefaction, numerical modelling of the beams was undertaken. The purpose of this modelling was to specifically assess how effective the HSM beams were in preventing or limiting liquefaction triggering in the soil between the beams, and as such supplemented the field testing

of the T-Rex shaker. The purpose of the HSM beams, as discussed above, is to develop a non-liquefying crust by confining the soil in between the beams to reduce γ and r_u. As such, the numerical modelling primarily assessed the development of γ and r_u generation of the soil between the beams (within the non-liquefying crust), although the effects beyond the "crust" itself were also assessed.

The numerical modelling included dynamic analyses with "total stress" (where pore pressure generation due to shearing was not included in the model, and "effective stress" (pore pressure generation due to shearing is included) constitutive models. The UBCHyst model (Naesgaard, 2011) was used for the total stress analysis while the PM4Sand model (Boulanger and Ziotopoulou, 2012) was used for the effective stress analysis. The total stress analysis enabled the relative differences of the γ for the natural soil and HSM beam soils to be examined while the effective stress analysis also allowed for the generation of r_u, further influencing the γ values in the latter stages of the dynamic simulations.

The finite difference program FLAC 7.0 (Itasca Consulting Group, 2011) was used for the 2D analysis. A soil profile representative of the conditions at Site 4 was used for all numerical analyses based on borehole, CPT and shear wave velocity (V_s) information (summarised in Figure 2a, b and c). Two configurations were analysed: an unimproved natural soil model and a model that included two rows of HSM beams. Figure 3 shows a schematic of the model set up near the ground surface for the HSM model. A surface surcharge of 10kPa was applied above the beams to approximate the surcharge from a typical residential building platform with a single storey dwelling. The beams were modelled as a square cross section with the same equivalent area as the production beams to simplify the model. A linear elastic constitutive model was used to for the beams.

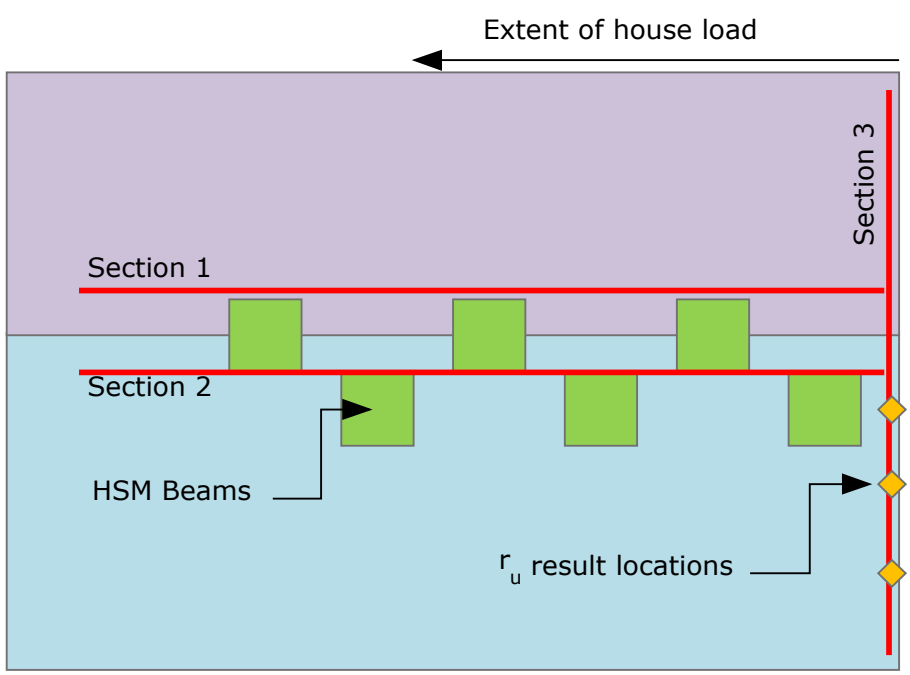

Figure 3. Schematic of the numerical model and the HSM beams relative to the ground surface and also relative to the 10 kPa residential building platform load.

For each model configuration, and for both the total and effective stress analyses, a suite of ground motions were used as dynamic input. These were selected to be representative of the controlling earthquakes at return periods of 25, 100 and 500 years. These return periods relate to the levels of ground shaking at the Serviceability Limit State (SLS), Intermediate Limit State (ILS) and Ultimate Limit State (ULS) that ordinary residential structures must be designed to meet (MBIE, 2012). The SLS, ILS and ULS ground motions were defined as M_w=7.5 equivalent PGAs of 0.13g, 0.20g and 0.35g respectively. However, specific earthquake scenarios were considered in selection of ground motions which included local (Christchurch) fault events, Canterbury Plains events and an Alpine fault event. The following earthquake records were selected: Darfield (2010) Canterbury Aero Club (CACS) recording station (used for the SLS, ILS and ULS events); Darfield (2010) Riccarton High School (RHSC) recording station (used for the SLS, ILS and ULS events); Christchurch (2011) CACS (used for the ULS event); Christchurch (2011) RHSC (used for the ULS event); Landers (1992) Joshua Tree (NGA ID 864) (used for the SLS, ILS and ULS events); Chi Chi (1999) TCU078 (NGA ID 1512) (used for the SLS and ILS events); and Denali (2002) Fairbanks – Geophysical Obs., CIGO (2110) (used for the SLS and ILS events). The RHSC and CACS recordings were de-convolved as required. Recordings were scaled to a target PGA for the various design motions, accounting for the various magnitudes using the Idriss and Boulanger (2008) magnitude scaling factors. In total, fifteen different ground motions were analysed, five for each level of shaking.

Results of the effective stress analyses are presented in Figure 4, in terms of maximum γ and r_u along the profile labelled Section 3 in Figure 3. The results are presented as the reduction (percentage change) in γ_{max} and r_u for the HSM beams model when compared to the unimproved model. Each trace represents the results for a single ground motion. The γ_{max} profiles clearly show a variation in behaviour between the soil within the zone of the HSM beams and also below the HSM beams. A consistent trend of reduced γ_{max} can be seen at all levels of shaking between the beams. This result supports the overall improvement concept for HSM beams, which is one of increasing the thickness of the non-liquefying layer by reducing γ in the soil between the beams. The results beneath the beams are somewhat more variable, however there is a trend of increasing γ_{max} over the 2m beneath the HSM beams. This result is considered most likely to be due to the presence of a stiffened layer above the zone of increased γ, and is perhaps akin to the soft storey phenomenon in structural engineering. Accordingly, it is considered unlikely that this result is specific to HSM beams, and is likely to be a phenomenon where any stiffened crust is included, for example a soil cement raft or gravel raft.

The γ_{max} results indirectly indicate the efficacy of the HSM beams in preventing or limiting liquefaction. The r_u results provide a direct measure of this effect. The percentage change in r_u results in Figure 4 are shown vs time since the commencement of strong shaking in each ground motion record, for the same point in time during the earthquake record for the unimproved and HSM beam model. The results presented are at three points on Section 3, as shown in Figure 3. One of the points is at 2 m below the ground surface (bgs) which is between the HSM beams and the other two points are 2.5 and 3.1 m bgs, below the HSM beams. The results show marked differences in performance at the various levels of earthquake shaking. At the SLS level of shaking the results consistently show that the build-up of r_u is prevented in the HSM beam model within and immediately beneath the beams. There is no significant difference in behaviour at the

deepest recorded level. The ILS results show a consistent reduction in r_u throughout the earthquake records between the HSM beams, a delay in r_u build up immediately beneath the beams and a small increase in r_u at depth. The ULS results show a delay in the build-up of r_u between the HSM beams, as well as a reduction in r_u in two cases. At depth the results are inconsistent but show an increase in r_u build-up for two of the selected ground motions. Overall the results indicate that the HSM beams tend to reduce r_u build-up between the beams, but that this improvement does not extend to all levels of shaking. At ULS levels of shaking, whilst the tendency to reduce r_u is present, it is not sufficient to prevent liquefaction.

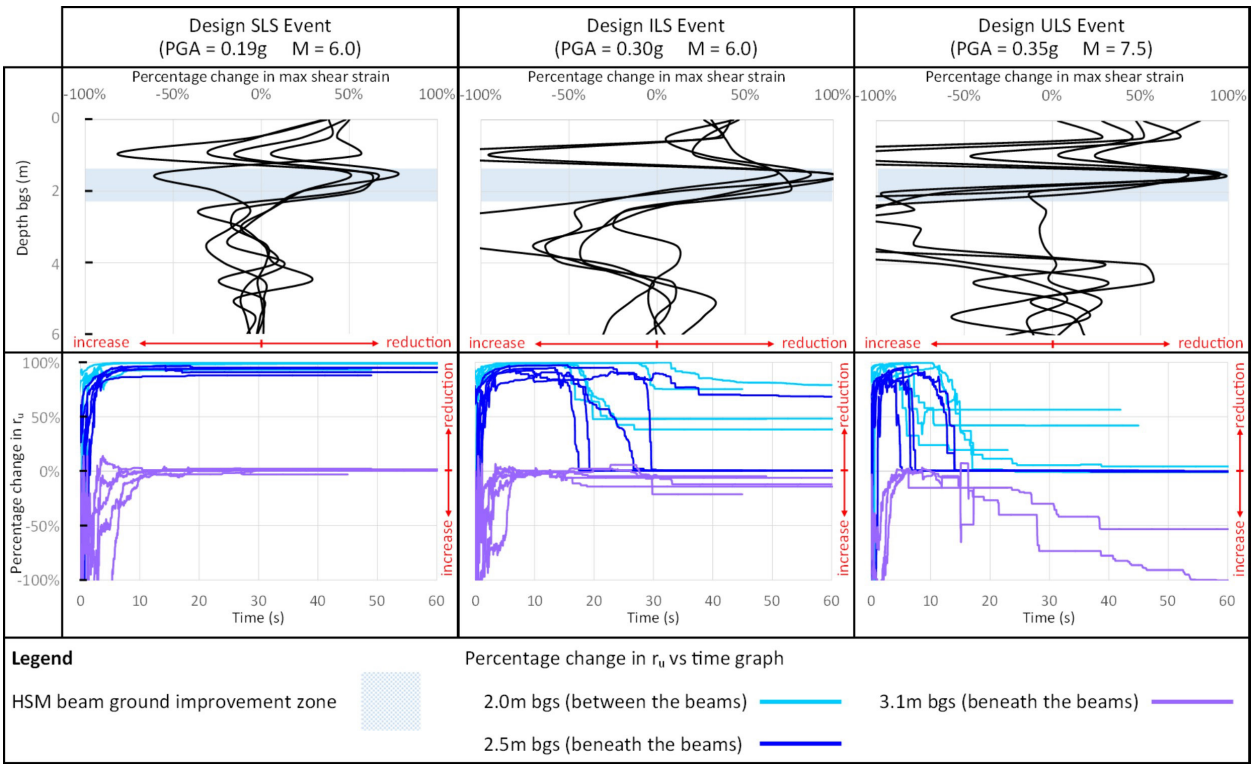

Figure 4. Percentage change in γ_{max} and r_u due to HSM beams for SLS, ILS and ULS scenarios.

The results in Figure 4 are near the centre of the HSM beams and residential house building platform. However, residential houses have a limited width, and the behaviour at the lateral extents of the structures is also of significance, particularly as more highly loaded foundations are typically found at the edge of the residential building platforms. Figure 5 presents the results of γ_{max} at the edge of the residential house building platform and HSM beam extents along Sections 1 and 2 (as shown on Figure 3), for the SLS ground motions only.

The results on the top row of Figure 5 show γ_{max} for the natural soil. The spike in γ_{max} across the profile is a result of the edge of the residential house building platform, indicates that the cyclic stresses are higher near the edges, resulting in localised liquefaction triggering at lower levels of shaking compared to the soil further away from the edge effects. The bottom row of Figure 5 shows the percentage change in γ_{max} due to the presence of the HSM beams. The results show a consistent reduction in γ_{max} between the beams compared to the natural soil case in general. There are some increases in γ_{max} immediately adjacent to the beams themselves, however this is

not considered significant due to the relatively thin zone over which this occurs. Overall the results indicate the beams are effective in reducing γ_{max} at the edge of building footprints.

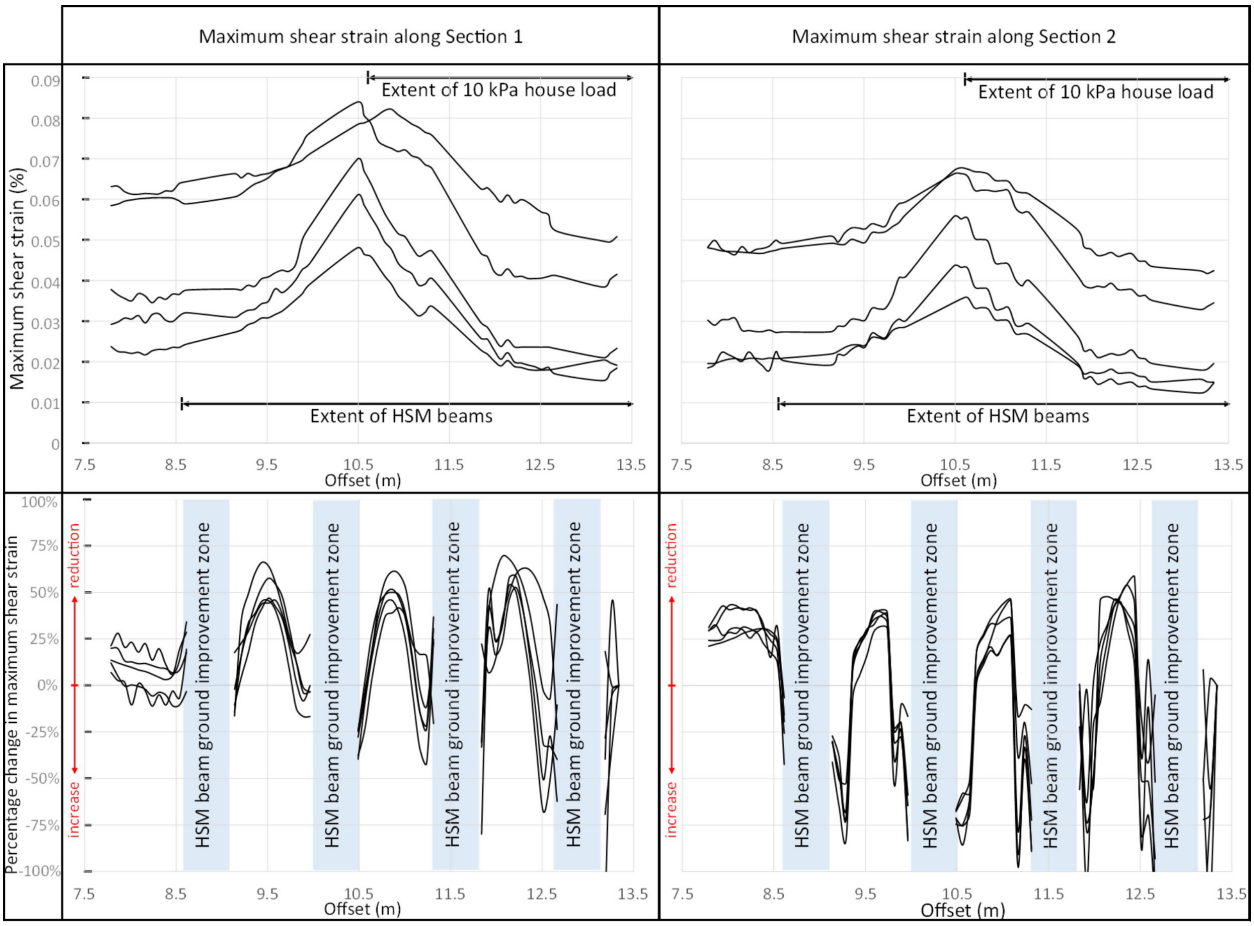

Figure 5. γ_{max} and the percentage change in γ_{max} near the edge of a house at SLS levels of earthquake shaking.

Discussion and Conclusions

Ishihara (1985) recognised that a thick non-liquefying crust overlying liquefying soils would reduce the consequences of liquefaction (i.e., sand boils, loss of bearing capacity and differential settlement). This was confirmed by the observations following the CES, where less structural damage occurred in liquefaction-prone areas containing an intact, relatively stiff non-liquefying crust with minimum thickness of approximately 3 m. The HSM beam ground improvement can be applied beneath existing structures to increase the non-liquefying crust thickness and hence decrease the liquefaction vulnerability. In-situ dynamic T-Rex shake testing was undertaken on HSM beam ground improvement panels to examine the liquefaction triggering and supplemented by dynamic numerical simulation.

Both the T-Rex shake testing and the dynamic numerical simulations showed that the HSM beams were effective in confining the interstitial soil that remains between the beams resulting in reduced γ and r_u at SLS and ILS levels of earthquake shaking. Therefore, at these levels of

shaking HSM beam ground improvement are effective in increasing the non-liquefying crust thickness and also increasing the crust stiffness (Wentz et al., 2015), decreasing the liquefaction vulnerability of the existing residential houses. At ULS levels of shaking, whilst the tendency to reduce r_u is present, it is not sufficient to prevent liquefaction and hence at these levels of shaking the HSM beams are unlikely to have much beneficial effect in increasing the non-liquefying crust thickness. However, there may still be some reduction in differential settlement at ULS levels of shaking due to the increase in crust stiffness as a result of the HSM beams (van Ballegooy et al., 2015b).

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