Soil-Structure Interaction Analysis of a Large Diameter Tank on Piled Foundations in Liquefiable Soil

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ABSTRACT

The results of seismic pile-soil-structure interaction (SSI) analysis of a large diameter tank supported on piles driven to a firm stratum are presented. The piles penetrate a sand layer susceptible to liquefaction. The layer was improved by driving steel inclusions into the ground. The liquefiable layer extended outside the footprint of the tank. SSI analyses were performed to demonstrate that liquefaction treatment was not required beyond the tank foundation footprint. Site response analysis was performed using SHAKE91 and the SSI analysis was performed using MTR/SASSI. Soil nonlinearities were accounted for using an equivalent linear model. The tank and fluid content were modeled using a lumped-mass stick model. The concrete base slab and insulation space between the steel and concrete base slabs were modeled using flat shell (plate) and spring elements, respectively. The piles were modeled using linear beam finite elements connected to soil at all pile nodes. The improved soil and the liquefaction zones were explicitly modeled using solid elements. For the liquefaction zone, the solid elements were assigned a degraded shear wave velocity value based on CPT data and residual strength relationships for sand. In addition to liquefaction effects the assumption that the bottom of the foundation slab is fully bonded to the soil was studied. It was found that the pile stresses for the unbonded case could be 50 percent higher than the bonded case.

INTRODUCTION

A proposed onshore liquefied natural gas (LNG) processing facility will include a large LNG tank. The tank consists of an inner cylindrical steel tank and an outer concrete containment tank with domed roof. The tank foundation slab is supported by 948 closed ended steel pipe piles driven to firm stratum at approximately 20 m depth. The piles have a nominal outside diameter of 0.6m and penetrate through a liquefiable sand layer prior to tipping into the bearing stratum. This required that the sand layer underneath the tank footprint be improved by an in-situ densification method to mitigate the potential for liquefaction. The analyses described in this paper were performed to demonstrate that it was not necessary to treat the sand layer outside the footprint of the tank.

The tanks were analyzed for the safe shutdown earthquake (SSE) for two soil cases covering the lower bound (LB) and upper bound (UB) properties and 7 ground motions. In this paper, only the results of LB soil are presented as the UB soils are not susceptible to liquefaction, which is the focus of this paper. To demonstrate the effects of SSI, the results are also compared with the fixed-base response of the tank. Additionally, the effect of bonding the foundation nodes to the soil is investigated. The methodology, modeling and analysis details and summary of the results and conclusions are presented.

LIQUEFACTION POTENTIAL

Stratigraphy. The stratigraphic profile in the LNG Tank Area consisted of (i) General fill to raise the existing grade from elevation +2.6 m (average) to +6.0 m above mean sea level, (ii) Unit 1 – mainly sands with some cementation from elevation +2.6 m to + 0.8 m above mean sea level.(iii) Unit 2a – mainly very soft to soft silt and clays from approximate elevation +0.8 m to -1.0 m above mean sea level, (iv) Unit 2b – mainly gravelly silty/clayey calcareous sands (average 15 % but as high as 30% fines) from approximate elevation -1.0 m to -4.0 m above mean sea level. Further details of the site can be found in (Galagoda et. al. 2016).

Ground Water. Ground water table was located approximately at elevation +0.9 m.

Liquefaction Assessment. In-situ standard penetration test results for the submerged Unit 2b soils ranged between 6 and 9 and the cone tip resistance (q_t) ranged between 3 and 5 MPa. Based on common correlations with CPT cone tip resistance values, the in-situ relative density of the Unit 2b soils ranged between 35% and 40%. Liquefaction screening was performed using recommendations made in NCEER (Youd et. al., 2001) under Operating Basis Earthquake (OBE) and Safe Shut Down Earthquake (SSE). The assessment concluded that Unit 2b sandy soils are susceptible to liquefaction.

Liquefaction Mitigation. Densification of the in-situ Unit 2b was adopted as a method to mitigate the liquefaction hazard. The concept consisted of driving 5 m long closed ended steel pipes (rigid inclusions) to the bottom of the Unit 2b layer in a group at certain spacing with the anticipation of achieving densification of the surrounding soils similar to what was expected when driving full-displacement closed-ended steel pipe foundation piles to support the LNG tanks.

The plan view of the pile and inclusion layout for the LNG Tank is shown in Figure 1. As shown in Figure 1, the spacing between the pile and the inclusion was approximately 2.0 m. The steel inclusions were driven to the desired depth using a mandrel.

Field Test Program. A test program was conducted to validate the densification process. Two field trial test zones were defined. The first trial zone was representative of the external rows of piles that are open ended and on a 2.3 by 2.3 m grid. This layout of the inclusions is shown in Figure 2. CPTs were conducted as part of baseline measurements to characterize the soil conditions prior to densification of trial program. A second test layout representative of closed ended piles representative of the central piles on a 3 by 3 m layout was also performed.

Target relative density to be reached to avoid soil liquefaction in layer 2b was then estimated using the NCEER approach (Youd et. al., 2001). The method was used backwards to estimate the required relative density value corresponding to a safety factor above (a) FS = 1.0 to avoid any major soil liquefaction in case of seismic event, (b) FS = 1.25 to avoid any excess pore pressure generation in case of seismic event. Corresponding target cone tip resistance required to meet the required relative density corresponding to F.S. of 1.0 and 1.25 were then estimated and plotted as shown in Figure 3. After raising the site to +6.0 m and driving all piles and inclusions into Unit 2b, several CPT tests were performed at distances between the piles/inclusions

The following conclusions were derived from the program. (a) The results from the tests don't give any specific trend regarding the densification of the ground with distance in the tested mesh., (b) For all the tests, the first 20 to 50 cm show a tip resistance less than required. This is counterbalanced by dissipation tests which show that this part of the layer is made up of a fine layering of silt and clay corresponding to the transition of Unit 2a to Unit 2b. Considering the nature of the soil and the limited thickness of this upper part of Unit 2b, it is considered not to be a global liquefaction hazard, (c) Densification improved the tip resistance up to more than 3 times the initial value, mitigating the risk for liquefaction, (d) All CPT tests were performed within 4 days after pile and inclusion driving. Since the soils contain some fines, some soil-setup is expected that shall also improve the densification.

MODELLING METHODOLOGY

The SSI analysis was performed using the Flexible Volume method (FVM) as implemented in the SASSI program (Lysmer, et.al. 1981). FVM is formulated in the frequency domain using complex frequency response method and finite element technique. The SASSI procedure does not require that the soil be modelled explicitly using finite elements unless the soil properties in certain zones differ from the free field soil. For this case, the improved soil layer underneath the tank and the liquefaction zone outside the tank footprint are explicitly modeled using solid elements. For the liquefaction zone, the solid elements were assigned a degraded shear wave velocity value based on CPT data and residual strength relationships for sand (Idriss, et.al. 2008). Further details of the FVM as related to nonlinear analysis using the equivalent linear method can be found in Tabatabaie and Tajirian (2017).

To account for the soil nonlinear behavior in the analysis the primary soil nonlinearity in the free field is first obtained from 1-D site response analysis using the SHAKE program. The resulting strain-compatible soil shear modulus and damping ratio are then inputted to the layered soil system used in the SSI model. In this application, secondary soil nonlinearity due to the SSI effects was ignored. A separate nonlinear lateral pile analysis using total seismic shear demand was performed to demonstrate that the soil is capable of resisting the forces without significant soil stiffness degradation.

Subsurface Characterization. The idealized soil profile consists of the seven layers described in Table 2. The water table is approximately 5.1m below ground surface. The low-strain soil profiles and properties for the free-field are summarized in Table 2. The soil profile under the tank was assumed to be the same as the free field except for Layer 2b. Densification of this layer resulted in increasing the free field shear wave velocity for this layer from 110 to 250 m/s. For the balance of the liquefiable section of Soil Layer 2b outside the tank footprint, a degraded shear wave velocity was used as described above. The properties used for the liquefied soil mass outside the Tank footprint under SSE are summarized in Table 3.

Table 2. Idealized Soil Profile and LB Soil Low Strain Properties

Soil Layer	Description	Approx. Thick. (m)	Shear Wave Velocity (m/s)	Poisson's Ratio	Mass Density (t/m^3)
Fill	Gravelly sand	3.5	180	0.33	1.94
Unit 1	Sand, silty sand, and gravel with some carbonate cementation	1.7	200	0.48	1.83
Unit 2a	Soft silt and clay	1.8	110	0.48	1.63
Unit 2b	Granular alluvial deposits	3.3	110	0.48	1.94
Unit 3a	Clay to sandy clay with stiffness increasing with depth	9	250	0.48	2.14
Unit 3b	Weak rocks (sandstone, siltstone, conglomerate)	14	500	0.48	2.24
Unit 4	Carbonate rocks	38	800	0.48	2.45

Table 3. Liquefied Soil Properties

Soil	Unit weight (MN/m3)	Poisson's Ratio	Shear Wave Velocity (m/s)
Liquefied	0.01901	0.48	21.2

Seismic Design Ground Motions. A probabilistic seismic hazard assessment (PSHA) was performed for the tank site to develop the input design spectra. The structure is analyzed for two design level earthquakes, the OBE (Operating Basis Earthquake) with a return period of 475 years; and the SSE (Safe Shutdown Earthquake) with a return period of 2475 years. This paper presents the results of the SSE analysis. The spectra were developed at the ground surface for a stiff soil site (site class D). Seven sets of acceleration time histories, each set comprising of two horizontal and one vertical component were selected. The time histories were spectrally matched to the 5 percent damped SSE design spectra. The peak ground accelerations for the SSE are 0.55 g in the x and y directions and 0.35 g in the z direction.

Model Development. A detailed finite element model of the tank, foundation slab, piles, improved soil layer and liquefiable layer were developed. The concrete outer tank and steel inner tank were modeled as stick models with flexural and shear stiffness, and with equivalent density and/or lumped mass at the nodes to model the mass. The base slab was modeled with plate elements with actual thicknesses in the central part and stiffened plates under the outer shell to reflect the shell stiffening effect. Fluid interaction is accounted for with convective and impulsive masses computed from Housner's theory and placed at the appropriate elevation to reflect the effects of both the pressures on the cylindrical tank shell and on the bottom slab. The SSI model schematic is shown in Figure 4. A quarter SASSI model was developed taking advantage of symmetry and is shown in Figure 5. The base slab was modeled using plate/shell elements and the piles were modeled with beam elements.

The improved soil portion of Soil Layer 2b between elevations -9.1m and -4.2m under the tank was explicitly modeled using solid finite elements and incorporated into the SASSI SSI model as part of the structure (see blue color zone in Figure 5). The liquefiable soil in Soil Layer 2b extends 1 radius outside the Tank foundation footprint (see brown color zone in Figure 5). This portion of Soil Layer 2b was also explicitly modeled using solid finite elements and incorporated into the SASSI SSI model as part of the structure (see Figure 5). The liquefiable soil block model uses a fine mesh to allow a passing frequency of 12.5 Hz. The strain-compatible shear wave velocity and damping of the liquefiable soil block were assumed to correspond to 3% effective shear strain as explained above. The resulting mesh consisted of 163,488 nodes, 32,573 interaction nodes, 7,073 pile elements and 69,252 soil block elements. To compare the results for the case with liquefaction with the case of no liquefaction, the analyses were repeated using a SASSI model without the solid elements outside the tank footprint (brown color in Figure 5). This model had 60,917 nodes, 14,603 interaction nodes and 18,252 soil block elements. The SSI analyses were performed using MTR/SASSI High Performance Computing (HPC) program.

Bonded vs. Debonded Foundation Case. Often when SSI analyses of tanks are performed the assumption is made that the foundation slab is fully bonded to the soil. This field condition is difficult to ensure when the foundation is supported on piles. Because the piles are end bearing on stiff soil formation, the self-weight of the tanks will be transferred for the most part to the piles as the soil around the piles near the foundation base slab settles over time. This will result in relatively small or insignificant frictional resistance mobilized at the bottom of the base slab. To assess the impact of debonding (i.e., frictionless base slab) on the tank and pile responses, SSI analyses were performed with both bonded and debonded base slabs. The effect on accelerations, forces and pile stresses are presented below.

SOIL-STRUCTURE INTERACTION ANALYSIS STEPS

Site Response Analysis. One-dimensional soil column analysis was performed using SHAKE91 to develop strain-compatible soil properties and input motions, as appropriate, for the SSI analysis of the LNG tank. Nonlinear soil properties consisting of G/G_{max} and damping ratio versus shear strain were developed for the various soil layers and are shown in Figure 6. Note that Unit 4 constitutes a rock formation, and is modeled as linear elastic. Two sets of analyses were performed – one using the free-field soil column and the other using the under-tank soil column – subjected to seven sets of H1 components of SSE input motions. The average strain-compatible soil properties from these analyses were calculated and used in the SASSI SSI analyses. It should be noted that the results of the under-tank soil column analysis are only used to estimate strain-compatible soil properties for the improved portion of Soil Layer 2b.

The design earthquake input motion was developed as a Site Class “D” surface motion. This would have required that the design time histories to be applied at the surface of the soil profile. While this requirement is acceptable for the upper bound (UB) soil case, it results in numerical inaccuracies when applied to the lower bound (LB) soil case due to the presence of two soft layers 2a and 2b which have degraded shear wave velocities less than 50 m/s. To investigate this condition the SHAKE and SASSI analyses were first performed by applying the time histories at the surface of the soil profile. This caused the maximum accelerations computed in SHAKE for layers 2a and 2b to exceed 1.2 g. This happens because the constrained input motion at the surface is not compatible for this profile requiring an unreasonably high acceleration below the soft layers to match the target surface acceleration. These very high accelerations at depth get transferred to the piles and result in tank accelerations much higher than the UB case. It is not normal to compute tank response that is higher for a softer soil profile. Because of this investigation it was decided to apply the motion at the bottom of the Soil Layer 2b as outcropping. The responses at the surface were obtained from the site response analysis and used as input to the SASSI SSI analyses, see Figure 7. Normal practice for SSI analysis (ASCE 4-16) requires that the selected seismic input motions be appropriate for the geological environment where the facility is located. Local subsurface conditions at the facility site should be considered in determining the seismic input motions. Additionally, when soft layers overly stiffer competent materials, similar to the LB profile, the motions shall be specified as outcrop motions at the elevation of the top of competent material; i.e. bottom of layer 2b. This results in a much more realistic seismic environment and avoids numerical inaccuracies.

Three-Dimensional SSI Analysis. The SSI analysis was performed using an advanced version of the SASSI program capable of analyzing models with large numbers of interaction nodes (MTR/SASSI HPC). Several cases were analyzed. The cutoff frequency for the LB soil case was 12.5 Hz. In this paper, the following cases are presented:

1. Tank with the base slab fully bonded to the soil, SSE LB Soil
2. Tank with frictionless base slab (debonded), SSE LB Soil
3. Tank with the base slab fully bonded to the soil, SSE LB Soil, with Liquefaction
4. Tank with the base slab on Fixed-Base (No SSI effects).

DISCUSSION OF ANALYSIS RESULTS

Maximum Acceleration Profiles. The x-direction average maximum acceleration profiles in the outer and inner tanks for the bonded case on non-liquefied and liquefied soil outside the tank are compared in Figure 8. Comparisons show no significant difference in tank accelerations between the liquefied and non-liquefied soil outside the tank footprint. Thus, leaving liquefiable soils untreated outside the tank footprint has minimal or no effect on the tank acceleration. The fixed-base acceleration profiles are also shown in Figure 8. As shown in Figure 8, considering SSI effects results in large reductions in the tank acceleration responses.

For the debonded cases the maximum accelerations were also calculated (not shown). In general, the results indicate no significant change in the calculated average maximum acceleration profiles in both the inner and outer tanks due to the assumption of soil debonding.

Maximum Global Forces and Moments at Bottom of Inner and Outer Tanks. The maximum global shear, and normal forces as well as the overturning moments were calculated at the bottom of the outer and inner tanks for all SSI cases. The average of the seven input time histories was calculated and the results are compared in Table 4. As shown in Table 4, there is no significant difference in the calculated base forces and moments between the bonded and debonded cases for the case with no liquefaction outside the tank. The forces for the bonded case with liquefaction are also compared in Table 4. The maximum forces with liquefaction are about 5 to 15 percent lower indicating that leaving liquefiable soils untreated outside the tank footprint has only a small effect on the global base normal and shear forces as well as the overturning moments at the bottom of the outer tank once liquefaction is mitigated under the tank footprint. The effect of liquefaction under the tank on tank response was not investigated as project requirements would not allow this condition.

Pile Forces and Moments. Table 5 shows comparisons of maximum values of total pile top shear forces for the full foundation model (948 piles) for all three cases versus the total

Table 4. Comparison of Total Base Shear Force, Normal Force and Overturning Moment at Bottom of Concrete Base Slab

Foundation	Liquefaction	X-Force (MN)	Y-Moment (MN-m)	Z-Force (MN)
Debonded	No	231.4	6,504.6	457.0
Bonded	No	230.3	6,458.9	464.6
Bonded	Yes	218.7	5,627.4	391.7

foundation seismic demand calculated by summing the inertia forces in the structure. As shown in Table 5, the maximum pile top shear forces (no liquefaction) that assume a frictionless base slab (debonded) are about 43% higher than the bonded case. This significant increase in pile top shear forces in the debonded case is to be expected since the lateral resistance provided by the base friction in the bonded case now must be resisted by the pile shear.

Table 5. Comparison of Maximum Pile Top Shear Forces and Total Seismic Demand

Liquefaction	Total Pile Top Shear Force (MN)		Total Inertia Shear Force (MN) (c)	Ratio of (b)/(a)	(c)-(a)/(c)	(c)-(b)/(c)
	Bonded (a)	Debonded (b)				
No	210	300	604	1.43	0.65	0.5
Yes	201	N/A	546	N/A	0.63	N/A

The maximum foundation seismic shear demand calculated for the case of no liquefaction outside the tank is 604 MN. The maximum value of the sum of the pile top shear forces is about 210 MN for the bonded case and 300 MN for the debonded case. Based on the above results, approximately two-thirds of the total seismic shear demand for the bonded case is resisted by soil. For the debonded case, about one-half is resisted by soil. It is therefore important that if bonded conditions are assumed it is necessary to show that there is enough soil capacity to resist this demand without significant soil yielding. Note that in SASSI the initial soil stiffness and linear elastic analysis does not account for soil yielding at the bottom of the base slab or around the piles. Table 4 also shows that for the liquefaction case, the total pile shear and total inertia are slightly lower than the non-liquefaction case. This observation is like the above findings.

To investigate the effect of liquefaction on the individual piles, the maximum axial and shear force, and bending moment profiles along the pile length due to input motions in the X-, Y- and Z-directions were calculated using the 100-40-40 seismic load combination rule for a representative pile in the outermost ring of piles, see Figure 9. This pile was selected because it is under the outer tank wall and resists the highest overturning effects. The reported values are averaged over seven sets of ground motions. As expected for an exterior pile and as this Figure shows the results of the liquefaction case control the stresses for the most part. The increase in combined dynamic loads for axial force, shear force, and bending moment are about 15%, 20-35% and 10-30%, respectively. The pile design was checked against these higher demands and was found to be acceptable. A separate nonlinear lateral pile analysis using total seismic shear demand was performed to demonstrate that the soil can resist the forces without significant soil stiffness degradation

CONCLUSION

This paper presented the results of SSI analyses performed on a large LNG tank supported on piles. The impact of liquefaction outside the footprint of the tank on the seismic response was investigated. Based on the calculated response of two models, with and without liquefaction the following conclusions were made:

- In general, there is no significant change in the acceleration responses of the inner and outer tanks, the total seismic demand at the bottom of the outer tank, inner tank, and base slab between two cases corresponding to liquefaction and no liquefaction outside the tank footprint. This indicates that leaving liquefiable soils untreated outside the tank footprint has no significant effect on the tank response, see Table 4 and Figure 8.
- The pile top seismic demand forces and moments are found to have increased for the outer piles along the perimeter of the tank foundation slab for the case when liquefaction is allowed outside the footprint of the tank, see Figure 9.
- Ignoring any frictional resistance between the base slab and soil in the SSI and lateral pile analyses (debonded soil case) to account for long term soil settlement and separation of the soil from the bottom of the base slab can result in significant increase in pile shear stresses (up to 50%) as compared to the bonded soil case.

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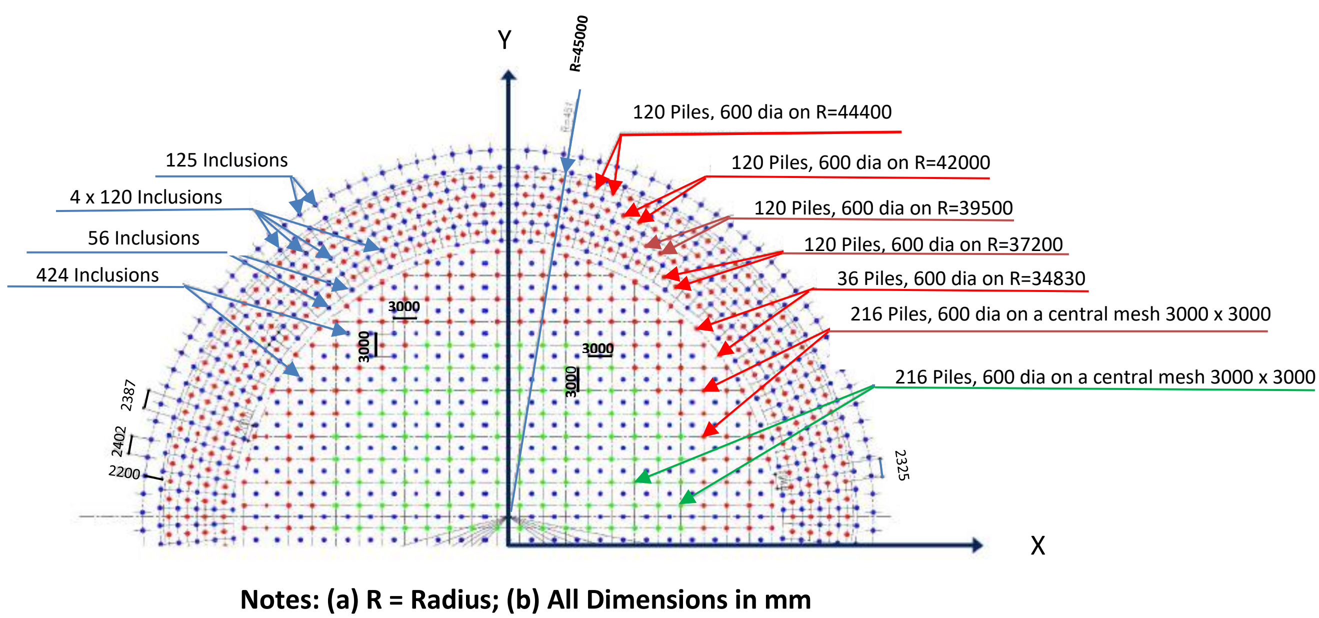

Figure 1. Plan View of Piling (Red/Green color) and Rigid Inclusion (Blue Color) Layout for the LNG Tank Foundation (Half Section Only)

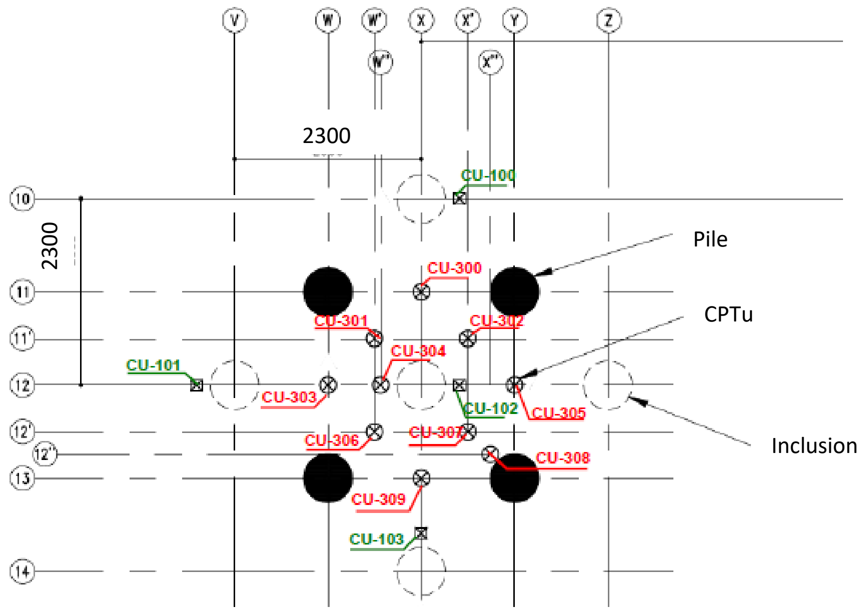

Figure 2. Pile and Inclusion Arrangement with CPTu Locations (Test Location 1)

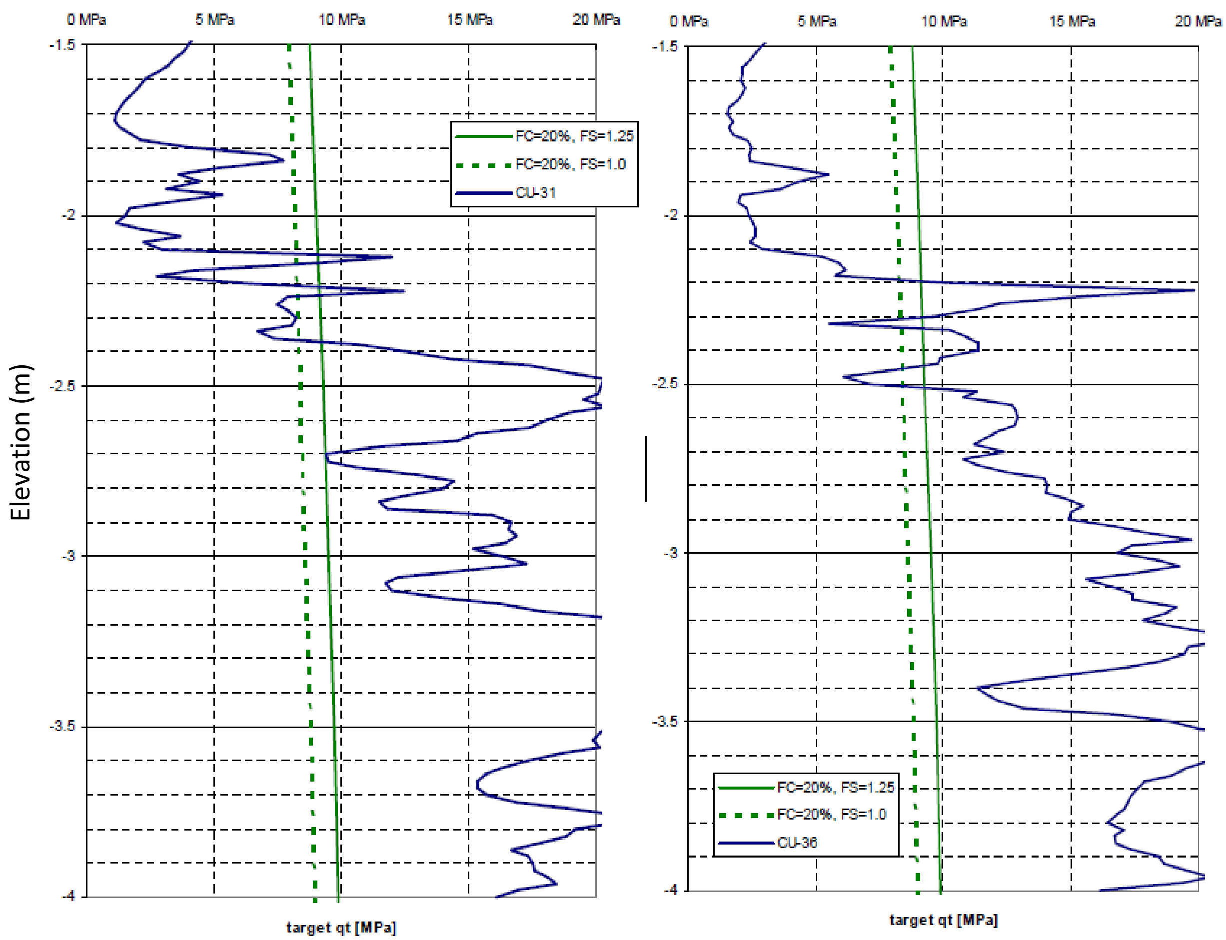

Figure 3 Typical Target q_t Vs. Post Densification Measured q_t for Layer 2B

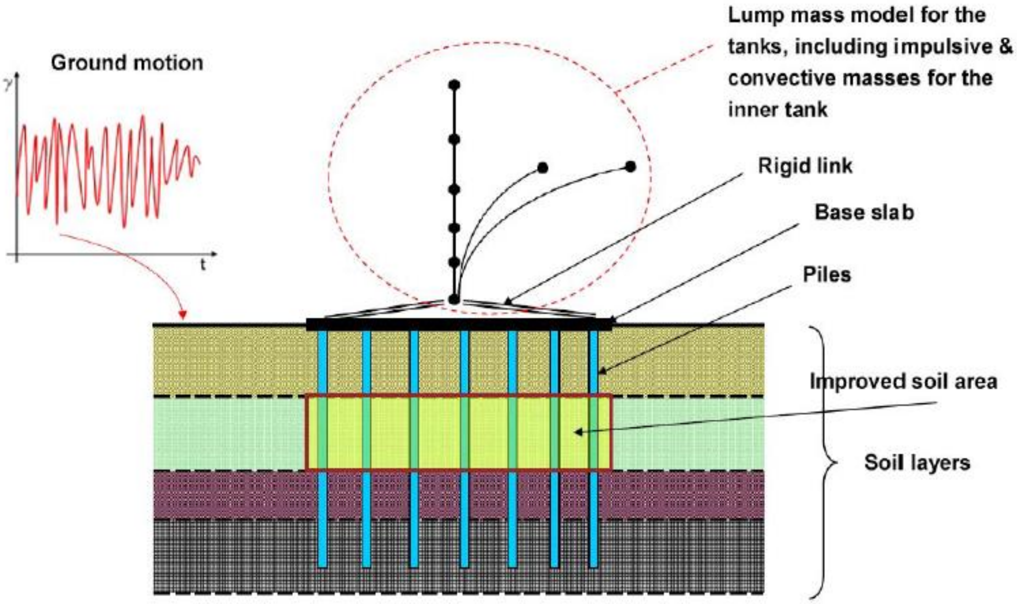

Figure 4. Schematic View of SSI Model of LNG Tank and Foundation System

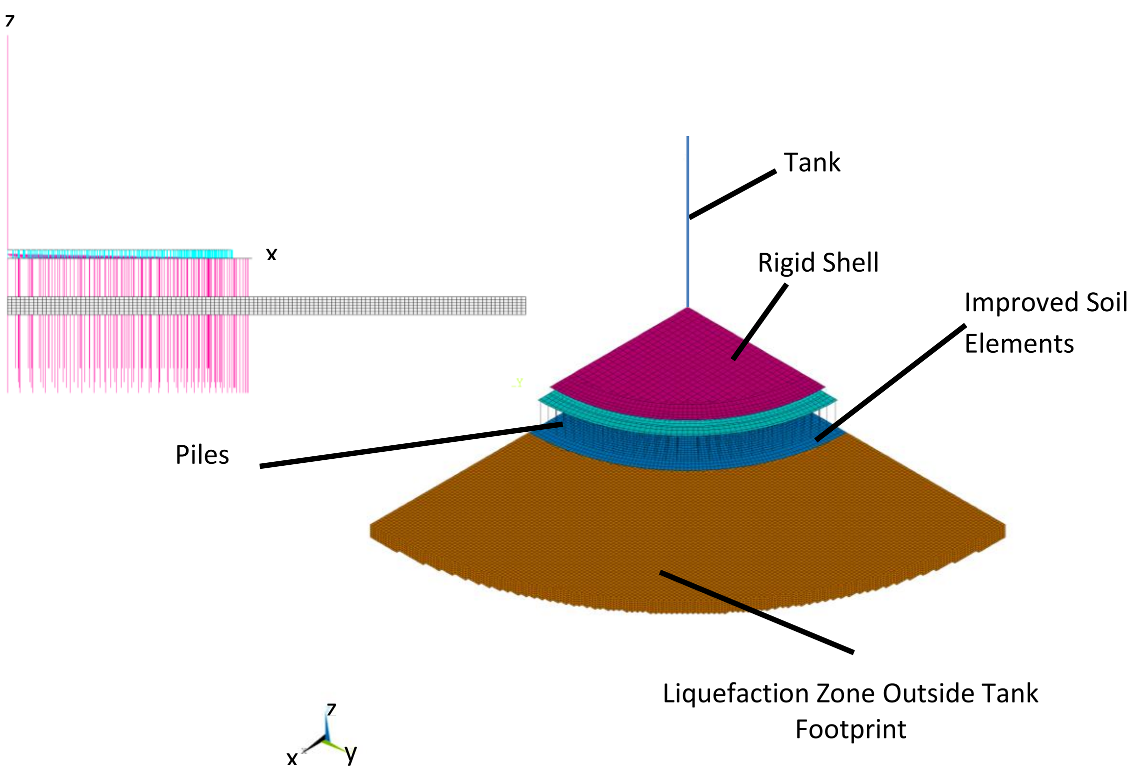

Figure 5. SASSI SSI Model.

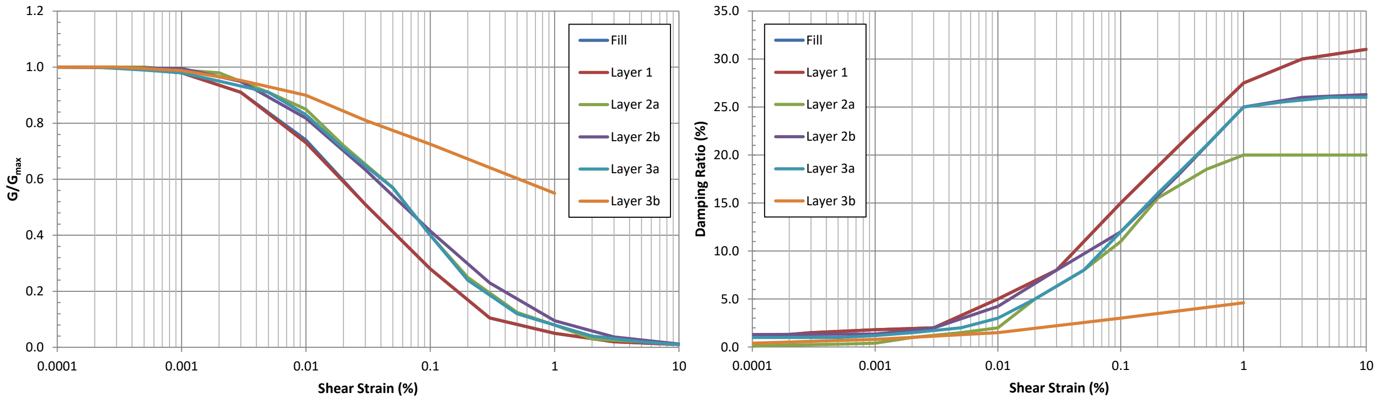

Figure 6. Strain-Dependent G/G$_{max}$ and Damping Ratio for Soil Layer

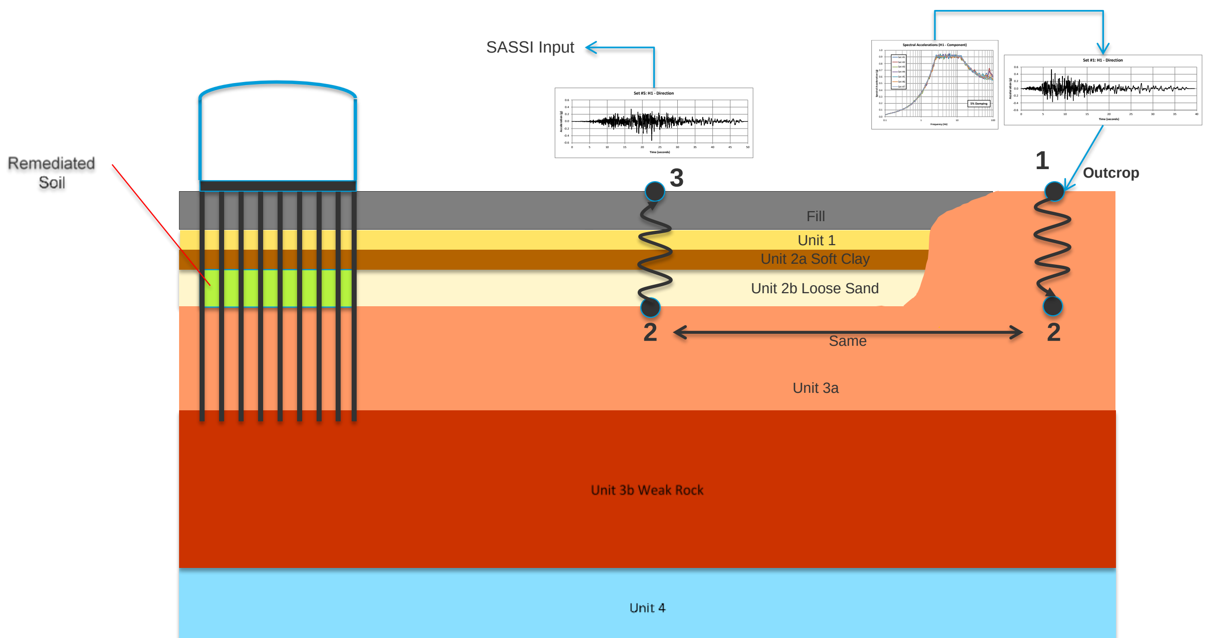

Figure 7. Site Response Analysis Procedure

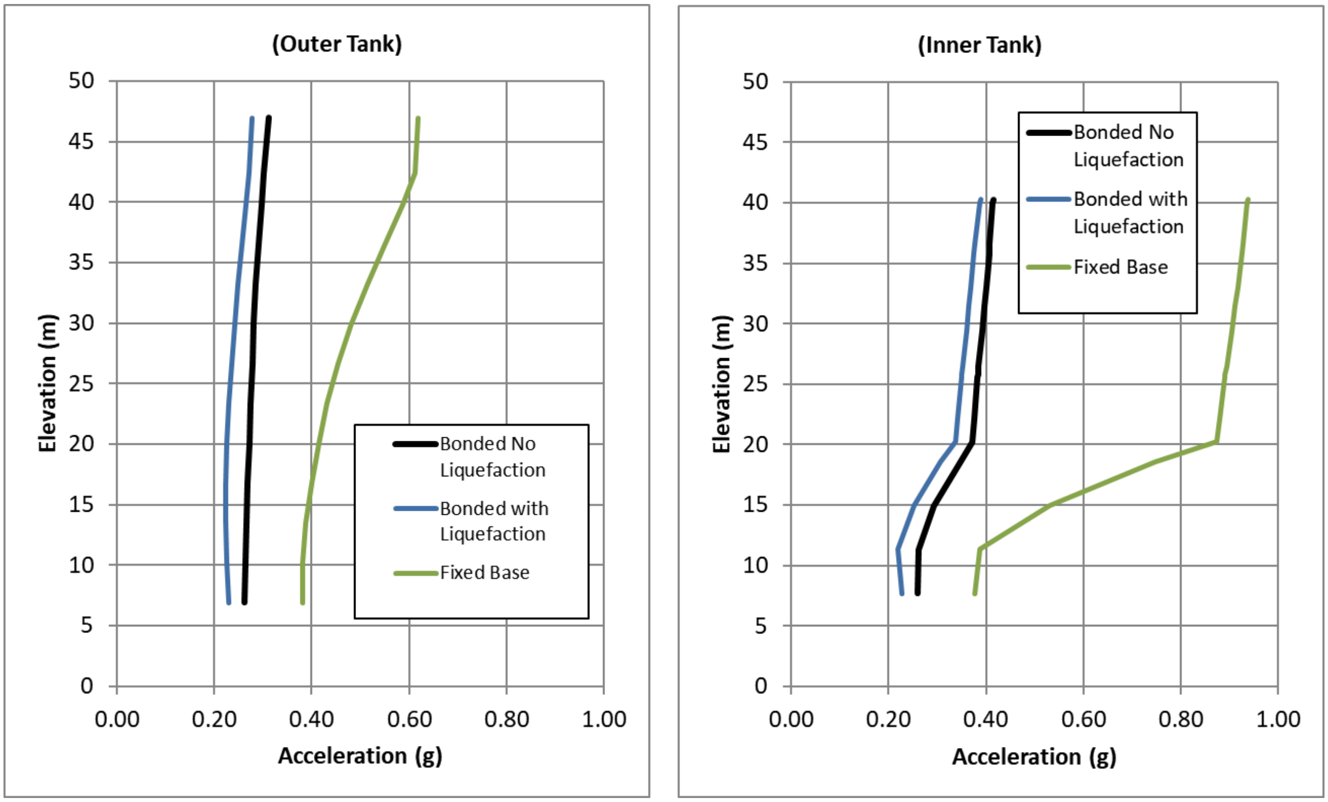

Figure 8. Maximum Horizontal Acceleration Profiles - SSE – X Direction

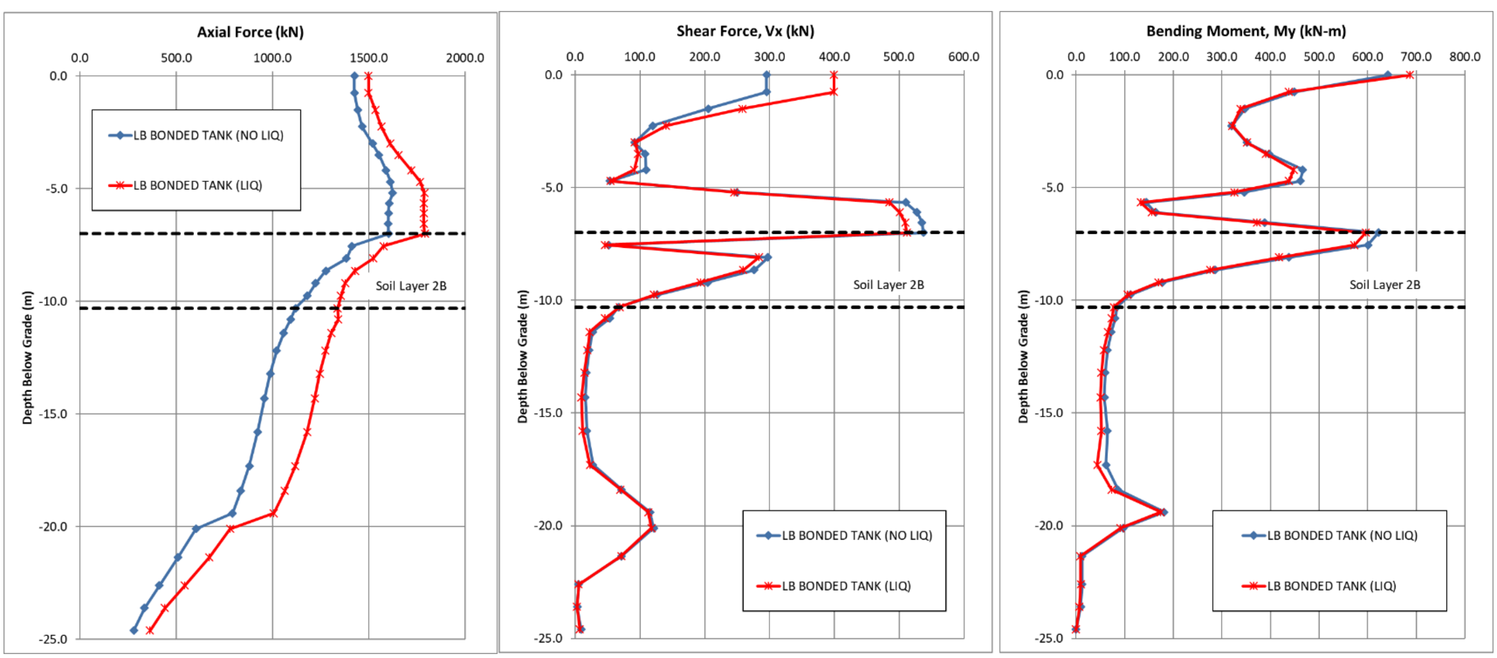

Figure 9. Comparison of Forces in Single Outer Pile (No Liquefaction vs Liquefaction)