Mitigation of Liquefaction Potential Using Rammed Aggregate Piers

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ABSTRACT

Trestle Glen is a mixed-use, mixed-income, transit-oriented urban reuse development on 6,880 square meters (1.7 acres) that now offers 119 units of affordable housing along with commercial and light industrial options adjacent to the Colma Bay Area Rapid Transit (BART) Station in Colma (San Mateo County), California. Subsurface soil at the Trestle Glen site includes artificial fill over potentially liquefiable, loose- to medium-dense silty to clayey sands, as well as soft silts and low-plasticity clays, extending to depths of about 7.6 to 9.1 meters (25 to 30 feet) locally along the historic creek alignments in the southeastern portion of the site. Groundwater is approximately 3.4 meters (11 feet) below the ground surface in this portion of the site. This paper presents a case study of the use of impact rammed aggregate piers (RAPs) to mitigate potentially liquefiable soil. The RAP system is a proprietary ground improvement method that involves driving a 30-centimeter (12-inch) diameter hollow mandrel with a 40-centimeter (16-inch) diameter rammer foot to the design depth. The hole is then backfilled with open-graded aggregate that is vibrated in 0.3-meter (1-foot) lifts by a dynamic impact hammer. This paper includes the results of a pre- and post-ground improvement Cone Penetration Test (CPT) program implemented to evaluate the post-ground improvement liquefaction and seismic settlement potential.

INTRODUCTION

The project is within a mixed residential, commercial, and light industrial area of Colma, California. The site is approximately 12.9 kilometers (8 miles) south of the City of San Francisco. The site is relatively flat with grades of approximately 39.6 to 40.5 meters (130 to 133 feet) above mean sea level. The development includes four levels of wood-framed construction over a podium. The podium foundation consists of spread footings with a slab-on-grade. The footing depths vary from 46 to 107 centimeters (18 inches to 42 inches) below grade.

Historic Conditions

The Colma formation underlies the majority of the site and consists of dense to very dense fine- to medium-grained Pleistocene sand with moderate amounts of clay and silt. The Colma formation has locally been incised by the historic alignment of Colma Creek and an unnamed creek, which are shown on the 1896 historic topographic map of the site included as Figure 1. The historic confluence of the two creeks seems to be located near the southeast corner of the property with relatively shallow groundwater and potentially liquefiable stream channel deposits, which are characterized locally along the historic creek alignments.

Figure 1. Historic Topographic Map – 1896
(USGS Map of San Mateo County, California 1:62,500 scale)

SEISMICITY AND FAULTING

The site is in a seismically active region. No known active faults are mapped at the property. The State of California zoned active faults near the site include the San Andreas, the San Gregorio, and the Hayward faults. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 10,000 years). Any one these faults could generate an earthquake capable of causing strong ground shaking at the site.

Table 1. Regional Faults and Seismicity

Fault	Approximate Distance from Site (km)	Direction from Site	Mean Moment Magnitude
San Andreas	2.9	Southwest	7.9
San Gregorio	9.0	West	7.3
Hayward	27.4	Northeast	7.1

SUBSURFACE CONDITIONS AND LIQUEFACTION HAZARD

Subsurface conditions encountered during our exploration at the site generally consisted of fill, stream channel deposits, and sands of the Colma Formation. The fill predominantly consisted of loose to medium dense silty sand, with some medium stiff to stiff sandy clay and sandy silt. The stream channel deposits generally increased in thickness from the west to east portion of the site with depths of approximately 10 feet in the western portion of the site and up to approximately 9.1 meters (30 feet) in the southeastern portion of the site. The stream channel deposits generally consisted of interbedded layers of loose- to medium-dense silty sand and sandy silt mixtures with localized layers of medium stiff sandy clay, very soft sensitive fine-grained soil, and dense sand. The site is underlain, at depth, by dense to very dense cemented sands of the Colma formation. Groundwater was encountered between 3.4 and 3.9 meters (11 and 13 feet) below the ground surface in the southeastern portion of the site at the time of our exploration in 2007.

Standard Penetration Test (SPT) and Cone Penetration Test (CPT) data collected during the geotechnical exploration were used to evaluate liquefaction resistance at the site. The CPT and SPT data was analyzed using procedures originally published in NCEER-97-002 and summarized by Youd et al. (2001). Based on the results of the liquefaction evaluation, there is a high liquefaction potential within the southeastern approximately one third of the site. The project team evaluated several alternatives for the mitigation of liquefaction potential. After evaluating various deep foundation systems such as driven piles, torque-down piles, and auger-cast piles, the project team selected a ground improvement system because of its cost and schedule effectiveness.

RAMMED AGGREGATE PIERS

Impact Rammed Aggregate Piers (RAPs) are a method of ground improvement that are constructed by first driving a sacrificial plate and 30-centimeter (12-inch) outer-diameter hollow mandrel, with a 40-centimeter-diameter (16-inch-diameter) rammer foot, into the soil using a large static force in addition to dynamic vertical impact energy that is produced by a high frequency vibratory piling hammer. After driving to the design depth or practical refusal, the hollow mandrel is withdrawn a short distance and acts as a conduit for the bottom-feed placement of open-graded aggregate. The aggregate used typically ranges from a 2-centimeter to 2.5-centimeter (¾-inch to 1-inch) crushed stone. Once the aggregate is placed inside the mandrel, the mandrel and rammer foot are lifted and then driven back down to form approximately 0.3-meter (1-foot) thick compacted lifts. A vibratory impact hammer compacts the aggregate with static crowd force and dynamic impact energy. Compaction and densification of the surrounding soil is enhanced by cyclic shear stresses produced by the vertically vibrating mandrel and rammer foot.

The installation of RAPs improves the soil by densifying the potentially liquefiable soil, providing shear reinforcement, and providing improved drainage of excess pore pressure generated during a seismic event. In addition, RAPs improve the composite shear strength developed as a result of the high friction angle offered by the RAPs and stress concentration to the stiffer piers, allowing the use of significantly higher bearing capacities for shallow foundations.

RAPs were installed to a depth of approximately 8.5 meters (28 feet) below the ground surface or practical refusal in the southeast portion of the site. The RAPs were generally spaced 2.1 meters (7 feet) on center for slab support and typically spaced 1.2 meters (4 feet) on center under spread footings. Around the perimeter of the building, additional RAPs were included to provide lateral confinement of RAPs supporting the perimeter foundations. The densification of the potentially liquefiable deposits was documented by pre- and post-RAP CPTs. The bearing capacity of the improved ground was documented by a modulus test conducted by the ground improvement contractor in accordance with ASTM D1143.

GROUND IMPROVEMENT TESTING PROGRAM

The purpose of the ground improvement testing program was to verify the effectiveness of the RAP ground improvement method in mitigating liquefaction potential through comparison of pre- and post-RAP CPT resistance. A test area was selected within the ground improvement area at the southeastern corner of the site. Our evaluation of liquefaction potential mitigation using RAPs was focused on the densification of the soil surrounding the RAPs (matrix soil) as measured by CPTs. Data collected from the pre- and post-RAP CPTs was used to calculate the Liquefaction Potential Index (LPI) of the matrix soil and residual liquefaction and seismic settlement potential, as it relates to tolerable settlement of the constructed project. RAP area replacement ratios are higher within the spread footing areas than under the slab areas. Therefore, CPT data for two post-RAP cases were evaluated: (1) matrix soil under a spread footing and (2) matrix soil under the slab-on-grade area. A total of three CPTs were performed within the selected test area as shown on Figure 2. One pre-RAP CPT and two post-RAP CPTs which included the collection of discrete push samples for laboratory testing were performed. The pre- and post-RAP CPT data are presented in Figure 3. Figure 2 shows the locations of the pre- and post-RAP CPTs. Pre-RAP CPT-1 was used as the benchmark to evaluate post-RAP CPT testing under a spread footing and slab-on-grade area.

Results of Testing

Pre-RAP CPT-1, Post-RAP CPT-2, and Post-RAP CPT-3 were performed at the locations shown on Figure 2. The real-time CPT data was used to determine appropriate depths at which the direct push soil samples were obtained. Post-RAP CPT-2 and post-RAP CPT-3 were performed approximately 4 weeks after RAP installation within this area of the site in order to better account for the time-dependence of ground improvement, as discussed in more detail subsequently. The post-RAP CPT data show significant increases in tip resistance within the depth of RAP ground improvement (the upper approximately 8.5 meters). Figure 3 shows a comparison of the normalized clean sand tip resistances for the pre- and post-RAP CPTs.

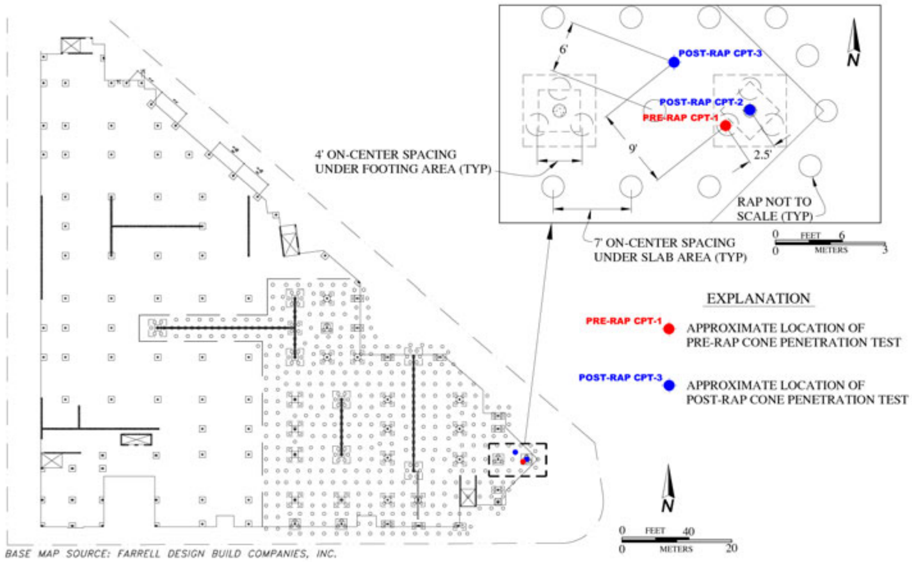

Figure 2. RAP Ground Improvement Plan

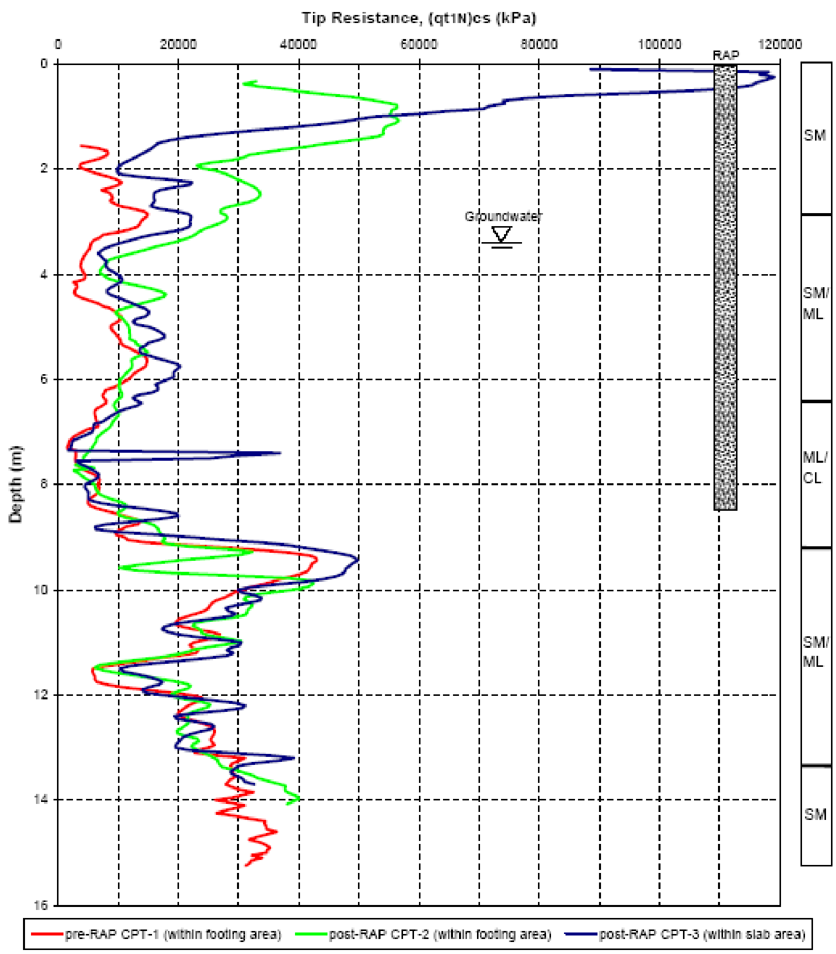

Figure 3. Pre- and Post-RAP CPT Tip Resistance

The calculated LPI and potential liquefaction settlements for the matrix soil do not account for the presence of the RAPs and the settlement control provided by the RAP elements. Calculated potential liquefaction settlements within the matrix soil were used to estimate expected potential composite (matrix soil and RAPs) settlements in areas beneath spread footings and slabs.

POST-IMPROVEMENT LIQUEFACTION POTENTIAL EVALUATION

The CPT data from the test program were used to evaluate post-ground improvement liquefaction and seismic settlement potential. The CPT data was used to evaluate liquefaction resistance using the computer software CLiq developed by Geologismiki. The procedure used in the software is largely based on procedures originally published in NCEER-97-002 and summarized by Youd et al. (2001). We analyzed the pre- and post-RAP installation CPT data using this procedure. We set the I_c cutoff at 2.60. We used the post-RAP CPT results to assess liquefaction risk and seismic settlement potential following site improvement. In order to evaluate the degree of improvement achieved by the RAPs, we compared the pre- and post-RAP installation test results.

Time-Dependent Resistance to Cone Penetration

Technical literature on ground improvement has documented significant time-dependent increases in CPT resistance following ground improvement. The time-dependent effects in sands have been attributed to delayed chemical reactions related to cementation and delayed dissipation of microgas bubbles (Charlie et al., 1992). To account for the time-dependent resistance to cone penetration we increased the post-RAP (4-week) normalized tip resistance by 33 percent to estimate CPT tip resistance at 10 weeks. We considered data from the studies by Mitchell and Solymar (1984) and Charlie et al., (1992) to select an appropriate factor for analyzing a 10-week projected normalized CPT tip resistance.

Laboratory Test Results

The laboratory testing of discrete direct push soil samples obtained during the advancement of the pre- and post-RAP CPTs resulted in fines contents predominantly in the range of 20 to 35 percent. Additional laboratory testing included Atterberg Limits testing to determine plasticity of the samples. The results indicate that the soils tested have low to no plasticity, with the majority of samples tested consisting of non-plastic silty sands. Less improvement is measured at the depth intervals with higher silt and clay contents when comparing the normalized clean sand tip resistances for the pre- and post-RAP CPTs shown on Figure 3. While less improvement may have occurred in these silty and clayey zones, they are likely less susceptible to classical liquefaction than sands with little to no silt and clay. The silty and clayey soil likely experience significant excess pore water pressures during seismic shaking which results in large cyclic shear stains sometimes referred to as seismic softening. As these excess pore pressures dissipate, volumetric strain occurs resulting in reconsolidation settlement of the affected soil layers. The reconsolidation settlement is essentially coincident with the ground improvement for sands. In the silts and clays, the reconsolidation settlement can be expected to occur more slowly over time.

Liquefaction Potential Index

To assess liquefaction hazard, we have expressed the pre- and post-RAP results using the Liquefaction Potential Index (LPI), as defined by Iwasaki et al. (1982). LPI is a relative hazard index, calculated on a point-by-point basis using the factor of safety against liquefaction, as a function of depth. LPI has been correlated to observed damage in existing liquefaction case studies and is a more appropriate indicator of risk than factor of safety alone. A summary of the pre- and post-improvement LPI is summarized on Figure 4. As shown on Figure 4, the post-RAP calculated LPIs are significantly lower than the pre-RAP calculated LPI. Calculated LPIs for the post-RAP CPT data using a 10-week increased cone penetration resistance show an additional reduction in risk for both post-RAP CPT-2 and post-RAP CPT-3.

Within the spread footing area where a closer RAP spacing was used, the reduction in LPI from the pre-RAP CPT-1 versus post-RAP CPT-2 shows a reduction from very high risk (LPI of approximately 17.8) to low risk (LPI of approximately 3.2). Based on the severity scale proposed by Iwasaki et al. (1982), the risk of liquefaction has been reduced from severe (LPI greater than 15) to not likely (LPI less than 5) within the spread footing area of the ground improvement zone.

Within the slab-on-grade area where a wider RAP spacing was used, the reduction in LPI from the pre-RAP CPT-1 versus post-RAP CPT-3 shows a reduction from very high risk (LPI of approximately 17.8) to high risk (LPI of approximately 9.6). Based on the severity scale proposed by Iwasaki et al. (1982), the risk of liquefaction has gone from severe (LPI greater than 15) to likely (LPI less than 15) within the slab area of the ground improvement zone. While the data suggests that some liquefaction may occur, the potential consequences of liquefaction will likely be limited to a small amount of acceptable settlement as discussed below.

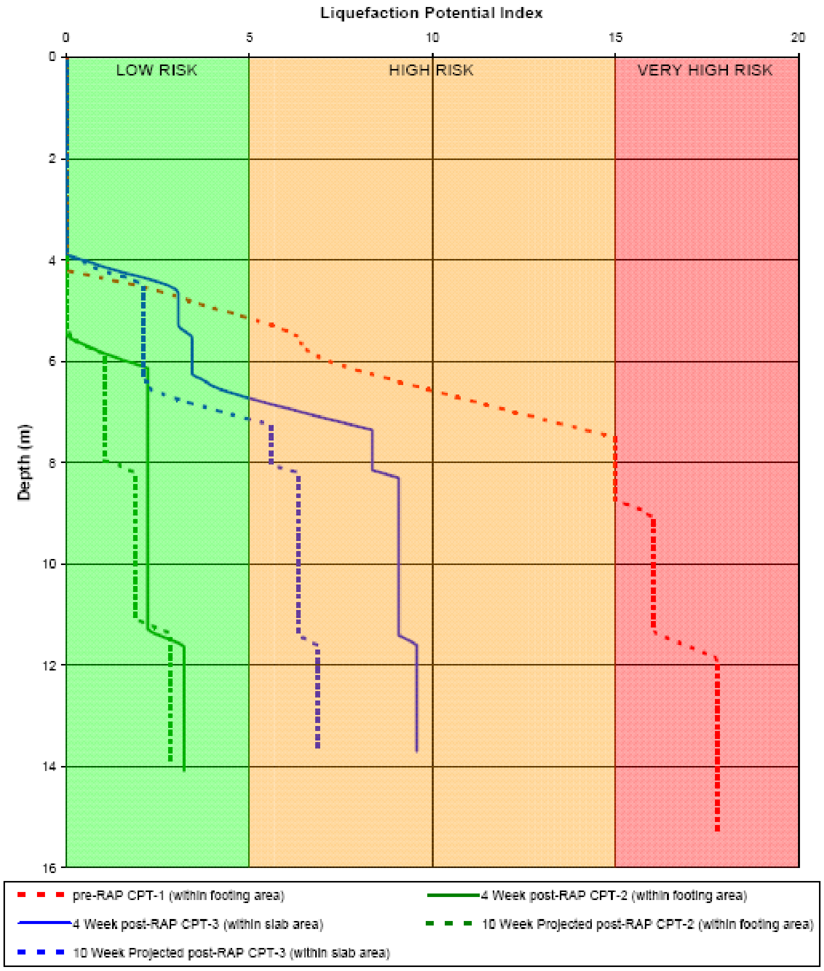

Figure 4. Pre- and Post-RAP Liquefaction Potential Index

Liquefaction Settlement

Figure 5 shows a comparison of the calculated potential liquefaction settlements of the soil surrounding the RAPS. The calculated potential soil settlements do not take into account the presence of the RAPs themselves which can be expected to experience lower volumetric strains, and hence less settlement, than the surrounding soil. Experience with ground improvement suggests that the actual settlement experienced by the structure will be reduced by the presence of the RAPs. As a result, the settlement estimates based on CPT testing are conservative estimates of potential seismic building settlements and actual settlements will likely be less.

As shown on Figure 5, calculated potential liquefaction settlement of the soil within the planned spread footing area is less than 2.5 centimeters (1 inch). The calculated potential liquefaction settlements of the soil within the footing area from pre-RAP CPT-1 versus post-RAP CPT-2 show a reduction of over 10 centimeters (4 inches). On the same basis, potential liquefaction settlement of the soil beneath the planned slab area is less than 7 centimeters (2¾ inches). The reduction in potential liquefaction settlements within the slab area from pre-RAP CPT-1 versus post-RAP CPT-3 shows a reduction of over 5.7 centimeters (2¼ inches).

Consideration of the test results at the 4-week post-ground improvement will generally lead to an over-prediction of future seismic settlement risk. In order to assess probable long-term site performance, time-dependent effects were also taken into consideration. Figure 5 includes a comparison of the measured 4-week CPT data and the projected 10-week CPT data. The calculated potential liquefaction settlements using 10-week increased cone penetration resistance data show little to no improvement for post-RAP CPT-2 and show a reduction of settlements on the order of 2 centimeters (¾ inch) for post-RAP CPT-3.

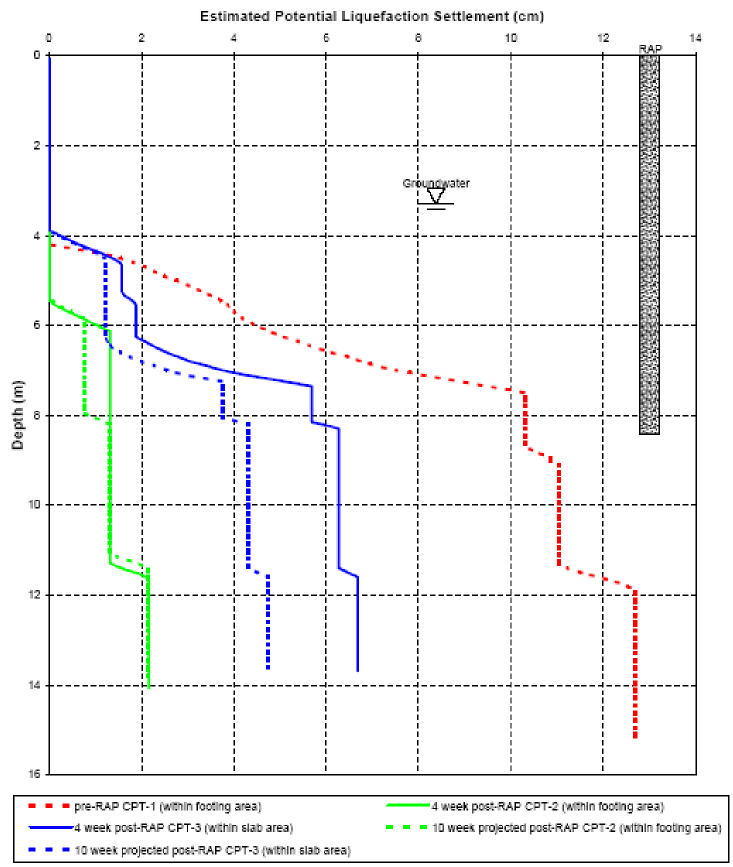

Figure 5. Pre- and Post-RAP Liquefaction Settlement of Surrounding Soil

In order to estimate the contribution of the RAPs in reducing predicted settlements, we employed a method used to evaluate static settlement reduction from stone column installation. We used the Priebe Method (FHWA, 1983) which considers area replacement ratio to estimate expected settlements following stone column installation. Using an average RAP diameter of approximately 61 centimeters (24 inches) and typical on-center spacing, we computed area replacement ratios of approximately 0.22 and 0.06 in the footing and slab areas, respectively. These replacement ratios were then used to calculate improvement factors. A friction angle of 45 degrees was assumed for RAP elements. Table 2 summarizes the predicted potential liquefaction settlements under spread footings and slab-on-grade areas based on the 10-week calculated potential liquefaction settlements of the soil surrounding the RAPs.

Table 2. Composite Settlement

	Post-RAP CPT-2 Spread Footing Area	**Post-RAP CPT-3 Slab-on-Grade Area**
Calculated Potential Matrix Soil Liquefaction Settlements at 10 weeks	2 centimeters (0.84 inches)	4.7 centimeters (1.86 inches)
Area Replacement Ratio	0.22	0.06
Improvement Factor	2.7	1.7
Predicted Potential Composite Settlement	0.8 centimeters (0.3 inch)	2.8 centimeters (1.1 inches)

CONCLUSION

Our evaluation of liquefaction potential mitigation using RAPs is conservatively focused on the densification of the soil surrounding the RAPs as measured during CPT probing and does not account for the effects of pore water pressure build up relief or increased site stiffness. Notwithstanding our data show that RAP installation has improved the site and has been effective at mitigating liquefaction potential to acceptable levels. The time-dependence of CPT penetration resistance was taken into account and indicates the potential for further liquefaction risk reduction with time.

Future case history studies should consider evaluating the effectiveness of the RAPs in reducing earthquake-induced cyclic shear stresses in the surrounding soil through the concentration of stresses to the stiffer RAP elements. In addition, the effectiveness of the RAPs in reducing excess pore water pressures is a mitigation method that merits further evaluation.

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