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DESIGN AND CONSTRUCTION OF COMPACTION GROUTING FOR FOUNDATION SOIL IMPROVEMENTS

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ABSTRACT

This paper presents the design and construction of compaction grouting work completed for a tank replacement project in Portland, Oregon. The project site is located along the west bank of the Willamette River. The subsurface soils at the project site were determined to be highly susceptible to soil liquefaction and lateral spreading under a design earthquake event per the building code. Compaction grouting was designed and constructed to strengthen the foundation soils supporting the new steel tank that is 115 feet in diameter and 40 feet in height.

The design of the compaction grouting was completed using the design guidelines outlined in ASCE/G-I Standard 53-10. Detailed quality assurance/quality control processes were implemented during grouting operations to account for the variability in soil conditions being grouted. Real time monitoring was also completed to evaluate the ground movement induced by the grouting process and its impact to adjacent structures and critical utilities. Pre- and post-grouting CPTs were completed to verify that the intended ground densification was achieved. A hydrostatic test was also completed with the tank filled with water. The tank foundation settlement under the hydrostatic test was found to range between ¼ to ¾ inches and met the acceptance criteria per API-650 and API-653 Standards.

INTRODUCTION

The Kinder Morgan Linnton Terminal is located at 11400 Northwest Saint Helens Road, Portland, Oregon, as shown in Figure 1. The project consisted of replacement of a century old tank LN-55021 located at the west end of the Linnton Terminal, approximately 400 to 500 feet west of the Willamette River. The old tank was a 32-foot high, 115-foot diameter steel tank supported on reinforced concrete ring foundations. Only the steel shell of the tank will be replaced with a height of approximately 40 feet (i.e. 8 feet higher than the old tank). The new steel shell is supported on the existing reinforced concrete ring foundations and steel tank bottom. The new tank and associated structural components weigh approximately 1,700 kips, and the tank will have an additional product weight (diesel) of nearly 21,500 kips when full.

Fig. 1. Vicinity Map

The key geotechnical design issue for the project is soil liquefaction and the associated settlement and lateral spreading under the design earthquake events. The on-site soils are found to be highly susceptible to liquefaction and that large lateral soil movement is anticipated within the tank footprint under the design earthquake events. Compaction grouting was implemented to reduce the amount of liquefaction-induced lateral movement (spreading) to an acceptable amount.

SITE CONDITIONS

Surface Conditions

Tank LN-55021 is located at the west end of the Linnton Terminal site. An approximately 10-foot high containment wall surrounds the tank on the southwest and southeast sides. These containment walls are tied to additional containment walls for other nearby tanks. Two nearby large tanks, along with associated pipes and equipment, are located immediately to the east and northeast of tank LN-55021. A series of smaller tanks are located to the northwest of the tank. Figure 2 shows the site plan and the approximate location of the cone penetration tests (CPT) completed for this project. Three CPTs (P-1, P-2 and P-3A) were completed during the design phase of the project. A fourth CPT (P-3B) was completed at the same location of P-3A after the compaction grouting was completed to evaluate the effectiveness of the grouting.

In general, the topography is flat in the vicinity of tank LN-55021 and throughout most of the Linnton Terminal site. Surface cover near the tanks consists of gravel fill. Figure 3 presents the photographs showing the surface conditions in the vicinity of tank LN-55021.

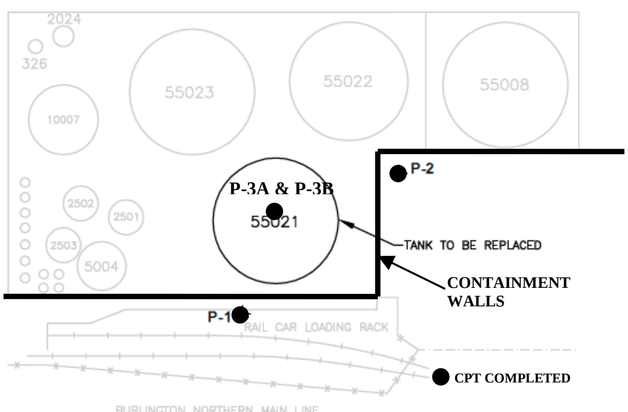

Fig. 2. Site and Exploration Plan

Fig. 3. Surface Conditions (looking west and southeast)

Subsurface Soil Conditions

The Linnton Terminal is located within the Portland Basin, which is part of the Willamette Valley physiographic province. The Willamette Valley is an elongate alluvial plain that was formed by uplift of the Coast Range to the west and the Western Cascades to the east and was subsequently filled with Pleistocene and Holocene alluvial sediment (Orr and Orr, 1999). The local geology at the Linnton Terminal site is mapped as Holocene-aged alluvial deposits, which are described as sand, gravel, and silt forming flood plains and filling channels of present streams. The underlying bedrock is mapped as Columbia River Basalt (Walker and MacLeod, 1991).

We explored subsurface conditions in the vicinity of the tank site by advancing three cone penetration tests (CPT) at the approximate locations shown in Figure 2. We advanced the CPT soundings to depths ranging from 49.5 to 56.5 feet below the ground surface (bgs), at which depths the CPT soundings met refusal.

Based on the CPT data we collected in the vicinity of the tank site, we interpret general subsurface conditions at the tank site as summarized in Table 1 below. This interpreted soil profile was used as the design soil profile for our soil liquefaction and lateral spreading mitigation design for this project. We estimated groundwater in the CPT soundings at depths ranging between 7 and 8 feet below the existing ground surface.

Table 1. Interpreted Subsurface Soil Conditions

Depth Interval (feet)	Soil Type
0 – 4	Med. dense to dense Fill
4 - 20	Med. stiff to stiff Clayey Silt/Silty Clay
20 - 30	Med. stiff Sandy Silt
30 - 50	Med. dense Sand/Silty Sand
50 - 56.5	Med. dense to dense Sand with Silt
56.5 +	Hard Basalt Bedrock

SEISMIC HAZARD AND DESIGN PARAMETERS

Regional Seismicity and Earthquake Source Zones

The Portland area is located near the convergent continental boundary known as the Cascadia Subduction Zone (CSZ), an approximately 650-mile-long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California. The CSZ is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The interaction of these two plates results in two potential seismic source zones: (1) the Benioff source zone, and (2) the CSZ interplate source zone. A third seismic source zone, referred to as the shallow crustal source zone, is associated with several northwest trending faults in the area.

According to the United States Geological Survey (USGS) Deaggregations website (USGS, 2008), the seismic hazard at the Linnton Terminal site is primarily due to the potential for a local shallow crustal earthquake to occur on the nearby Portland Hills fault. Large, long-duration interface subduction zone earthquakes occurring within the Cascadia Subduction Zone (CSZ) as well as deep, intraslab earthquakes occurring within the subducting Juan de Fuca plate may also affect the site; however, these earthquakes would occur at a much greater distance from the site than the Portland Hills fault. Therefore, near-source shallow crustal earthquakes occurring along the Portland Hills fault would result in higher ground motions at the site and control the seismic hazard.

Two design earthquake events are considered for this project. The first is the design earthquake event per Oregon Structural Specialty Code and the 2006 International Building Code (IBC). The second is the scenario earthquake that is associated to the nearby Portland Hills Fault. Table 2 below presents the seismic design parameters for the two design earthquake events considered for this project.

Table 2. Target Rock Outcrop UHS

Design Earthquake	Magnitude	Peak Ground Acceleration (g)
IBC Code Event	9.0	0.24
Portland Hills Fault	7.0	0.66

Notes:
a) Magnitude is taken as the modal event per 2008 USGS seismic deaggregation results.
b) Design PGA is taken as S_{ds}/2.5 per Oregon Structural Specialty Code.

SOIL LIQUEFACTION ANALYSIS

Liquefaction potential of the site soils were evaluated for the code design earthquake and the Portland Hills Fault event using subsurface data and information obtained from the CPTs. We evaluated liquefaction potential using the simplified method proposed by Youd, et al (2001). Figure 4 presents the factors of safety against liquefaction for the existing conditions under the IBC code and Portland Hills Fault events, respectively.

LATERAL SPREADING ANALYSIS

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of surface soils are displaced relative to adjacent blocks. Lateral spreading also occurs as blocks of surface soils are displaced toward a nearby slope or free-face by movement of the underlying liquefied soil. The Willamette River northeast of the site represents a free-face condition. The tank site is located approximately 400 to 500 feet from the top of the free face.

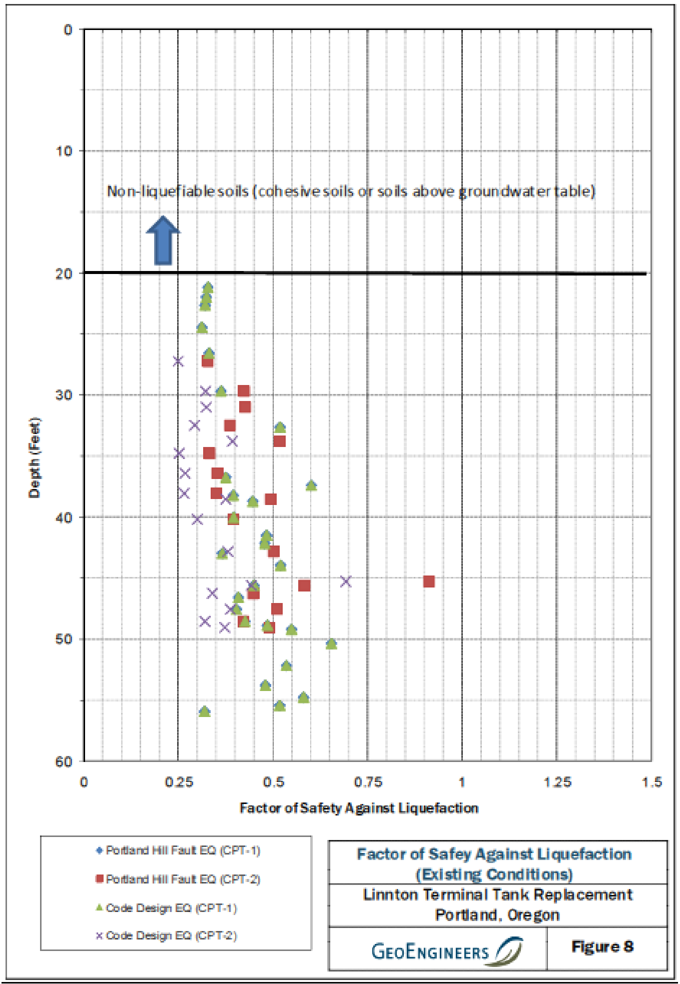

Fig. 5. Liquefaction Factors of Safety

The evaluation of lateral spreading at the site was initially completed using Youd's MLR simplified method, as a screening analysis. The results of the simplified method indicated that the site is susceptible to lateral spreading movement. Based on the results, additional analysis were completed to refine the amount of lateral spreading deformation that may occur during the seismic design events considered for this project.

Slope Stability and Newmark Analyses

Slope stability and Newmark analyses were completed to refine the lateral spreading deformation anticipated at the site under the design earthquake events. Slope stability analyses were completed using the computer program SLOPE/W (GEO-SLOPE International, Ltd., 2005). SLOPE/W evaluates the stability of the critical failure surfaces identified using vertical slice limit-equilibrium methods. This method compares the ratio of forces driving slope movement with forces resisting slope movement for each trial failure surface, and presents the result as the factor of safety.

Based on the subsurface conditions encountered in the explorations, and the results of soil liquefaction analysis, the representative engineering properties of the soil units under the seismic conditions were developed. Engineering properties of the soils not susceptible to liquefaction were developed using the guidelines presented in the National Cooperative Highway Research Program, NCHRP Synthesis 368 (Mayne, 2007) using the CPT data. For soils susceptible to liquefaction, the post liquefaction residual shear strength of the soils was estimated using the relationships developed by Idriss and Boulanger (2008).

Figure 5 presents the most critical failure surface that will impact the stability of the tank and to estimate permanent deformation of the identified critical failure surfaces under seismic loading conditions.

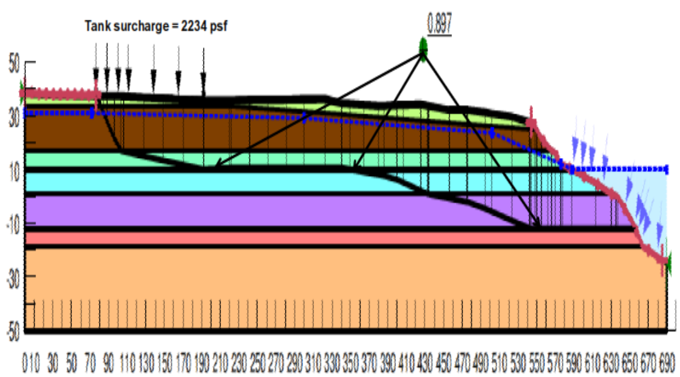

Fig. 5. Critical Failure Surface

Newmark analyses were completed using the computer program developed by Jibson and Jibson of USGS (Open File Report 03-005) using the rigorous rigid block method. The yield acceleration values calculated for the critical failure surfaces are used to estimate permanent lateral soil movement under the design earthquake time histories using the Newmark analysis method. The yield acceleration, which is defined as the ground acceleration that will cause a failure surface to start yielding or moving (i.e., FS = 1.0), were computed from our slope stability analyses.

Based on the results of the slope stability analyses, the factor of safety for the existing conditions after both design earthquake events is less than 1.0, suggesting that a lateral spread flow failure is likely during and after a design earthquake event if the subsurface soils liquefy.

A total of 97 earthquake time histories recorded at soft soil sites with a magnitude between 6.0 and 9.0 were selected for use in the Newmark analyses for the building code design earthquake event. All of the selected earthquake records were scaled to the design PGA of 0.24g. For the Portland Hills Fault event, a total of 48 earthquake time histories recorded at soft soil sites with a magnitude between 6.0 and 7.0 were selected for use in the Newmark analyses. All of the selected earthquake records were scaled to 0.66g to match the design PGA value of the Portland Hills Fault Earthquake event.

Figures 6 and 7 present the results of the Newmark analyses completed for the existing conditions under both the building code design earthquake event and the Portland Hills Fault earthquake event, respectively. As shown in Figure 6, the mean soil displacement for the building code design earthquake event is estimated to be more than 11 feet (3.3 m). The mean displacement of the critical slip surface is estimated to be more than 20 feet (6.2 m) for the Portland Hills Fault earthquake event, as shown in Figure 7.

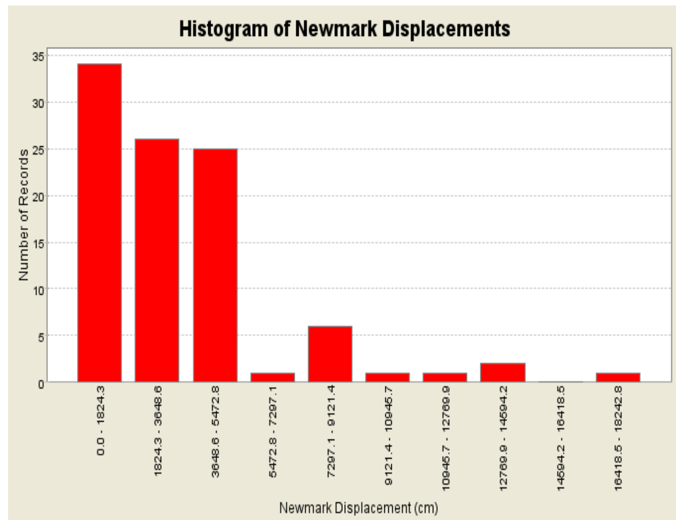

Fig. 6. Estimated lateral displacement (Existing Conditions, IBC Code Event)

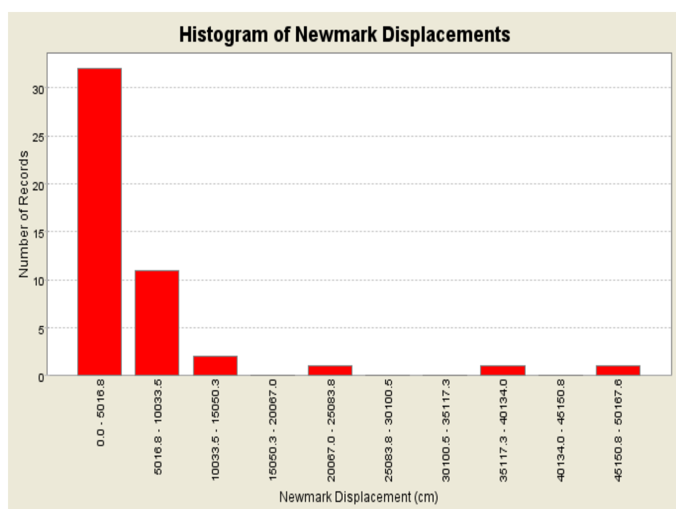

Fig. 7. Estimated lateral displacement (Existing Conditions, Portland Hills Fault Event)

COMPACTION GROUTING DESIGN

The results of the liquefaction and lateral spreading analyses indicated that the new tank will likely experience an excessive lateral deformation under both the design earthquake events. Ground improvement was recommended to strengthen the foundation soils and mitigate the liquefaction and lateral spreading hazards at the tank site.

A feasibility study was completed to evaluate several ground improvement alternatives to account for operational, constructability and environmental constraints. The compaction grouting option was identified as the most cost effective alternative. Operational constraints included close proximity to the containment walls and surrounding tanks and the need to minimize impact to operations during construction. Constructability constraints included limited site access. The environmental constraints included minimizing the exposure of potentially contaminated subsurface soils.

Compaction grouting is a process where low slump grout is pumped under pressure into the ground to be treated. The grout is typically injected from the bottom up in stages. The subsurface soil is displaced and compacted as the grout mass is pumped in the ground. In addition to the densification effect, the grout injected into the ground also increases the overall stiffness and shear strength of the treated soil mass. Another advantage of the compaction grouting program is the reduction of the cyclic shear stress in the treated soil mass (Baez and Martin, 1993).

The design of the compaction grouting was completed using the design guidelines outlined in ASCE/G-I Standard 53-10. The main objective of the compaction grouting design was to increase the post-liquefaction residual strength of the soils susceptible to liquefaction in order to reduce the lateral displacement of the tank foundation to a tolerable amount. Based on the evaluation of the structural engineer, the maximum tolerable lateral displacement of the tank foundation was estimated to be about 24 inches (61 cm).

Determination of the Compaction Grout Replacement Ratio

The results of our soil liquefaction analysis show that in order to reduce the lateral displacement of the tank, the soils that are highly susceptible to liquefaction at depths between 20 to 60 feet will need to be improved. The degree of improvement is dependent on the required level of densification and strengthening achieved by injecting the required grout volume in the ground. The grout volume is expressed in terms of grout replacement ratio, which is defined as the ratio of the injected grout volume to the volume of the treated soils.

In order to determine the required compaction grout volume, an iterative process was used in which:

1. The extent of the ground improvement zone and a trial compaction grout volume with an assumed minimum compressive strength is selected;
2. The degree of densification by injecting the compaction grout volume is determined using the design procedure outlined in ASCE/G-I Standard 53-10;
3. Engineering properties of the treated soils that include the effects of the compaction grout injected in the ground are determined; and
4. The lateral displacement of the tank foundation is then computed by completing slope stability and Newmark analysis using the improved engineering properties of the subsurface soils.

If the lateral displacements of the tank foundation under both design earthquake events are calculated to be less than 24 inches, then the selected improvement zone and compaction grout volume is appropriate. If the lateral displacements of the tank foundation under either of the design earthquake events are more than 24 inches, a larger improved zone and/or higher compaction grout volume will be selected and the process is repeated.

Figure 8 presents the extent of the compaction grout zone selected for the project, along with the critical failure surface for comparison purpose. In general, the tank foundation soils below depth of 20 feet that are susceptible to liquefaction will be improved. The compaction grout zone was extended to a depth of 10 feet to account for the potential variability of the soil conditions across the tank footprint. In addition, stopping the grouting at depth of 10 feet also provides the overburden stress that is needed for an effective grouting process.

Based on the results of the analysis, a compaction grout replacement ratio ranging from 2 to 9 percent as presented in Figure 9 was determined to be the optimum design for the project. The minimum compressive strength of the grout was determined to be 500 psi. The grout points were installed in triangular patterns with center-to-center spacing of about 10 feet.

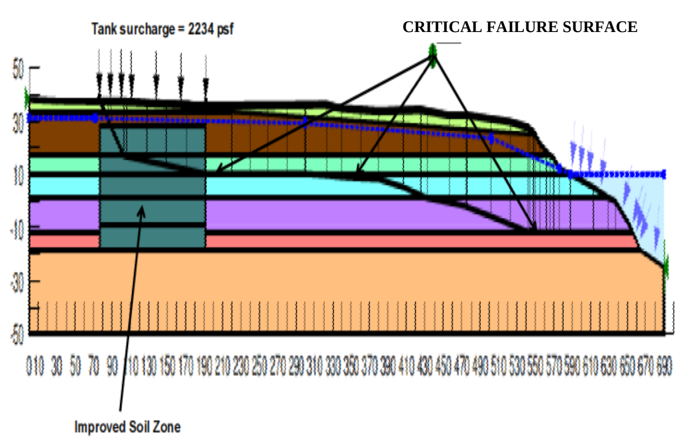

Fig. 8. Ground Improvement Zone

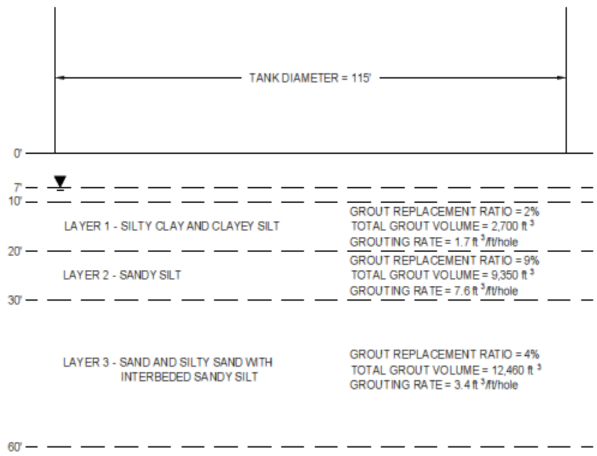

Fig. 9. Design Grout Replacement Ratio

Based on the results of the slope stability analyses completed using the improved soil properties, the yield acceleration computed for the critical failure surface identified for the post-grouting conditions was computed to be 0.20g. The improved soil zone was modeled using the weighted average strength of the grout and the soils with increased relative density.

The same suite of earthquake time histories used in the Newmark analyses for the pre-grouting were also used in the Newmark analyses for the improved conditions under both design earthquake events. Figures 10 and 11 present the results of the Newmark analyses completed for the improved conditions under both the building code design earthquake event and the Portland Hills Fault earthquake event, respectively. As shown in Figure 10, the mean soil displacement for the building code design earthquake event is estimated to be less than 1 inch (2.54 cm). The mean displacement of the critical slip surface is estimated to be about 15 inches (38 cm) for the Portland Hills Fault earthquake event, as shown in Figure 11.

The effect of the compaction grouting was also evaluated using the in-situ state and relative density (Shuttle and Jefferies, 1998) computed based on the CPT data. Figure 12 shows the in-situ state and relative density of the sandy soils for both the pre- and post-grouting conditions. The CPT values for the post-grouting conditions were estimated based on the increase in density of the soils as a result of injecting the 9 percent grout replacement ratio in the ground.

As shown in Figure 12, the relative densities of sandy soils computed for the pre-grouting conditions indicate that they are highly susceptible to liquefaction under even a small to moderate earthquake event. Upon completion of the compaction grouting, we estimated that the sandy soils would be densified to medium dense to dense state, which are still likely to liquefy under a large earthquake event, such as the Portland Hills Fault event.

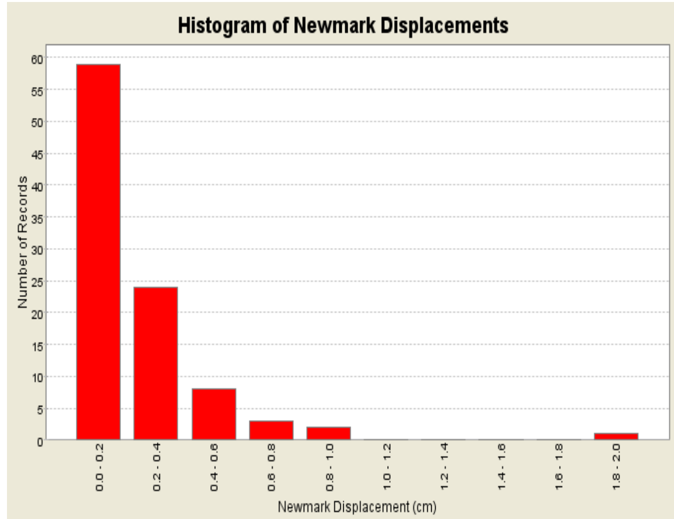

Fig. 10. Estimated lateral displacement (Improved Conditions, IBC Code Event)

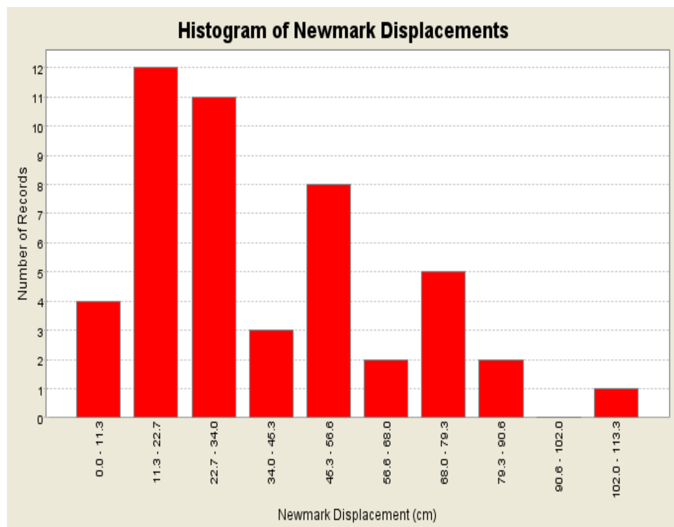

Fig. 11. Estimated lateral displacement (Improved Conditions, Portland Hills Fault Event)

Although the liquefaction hazard is not completely mitigated for the large earthquake event, the foundation soils were densified adequately such that the in-situ state of the soils is shifted from loose to very loose state (open symbols, ψ~0 to -0.10, contractive to lightly dilatant) to medium dense to dense (filled symbols, ψ~-0.05 to -0.20, lightly dilatant to dilatant). The change of the in-situ state would greatly reduce the permanent deformation of the soils, which is consistent with the results of our slope stability and Newmark analyses.

COMPACTION GROUTING CONSTRUCTION

Detailed plans and specifications for the compaction grouting program were also developed for construction that account for operational, constructability and environmental constraints identified at the site. The specifications also included detailed quality control and quality assurance measures implemented during construction to ensure that the design intents were met.

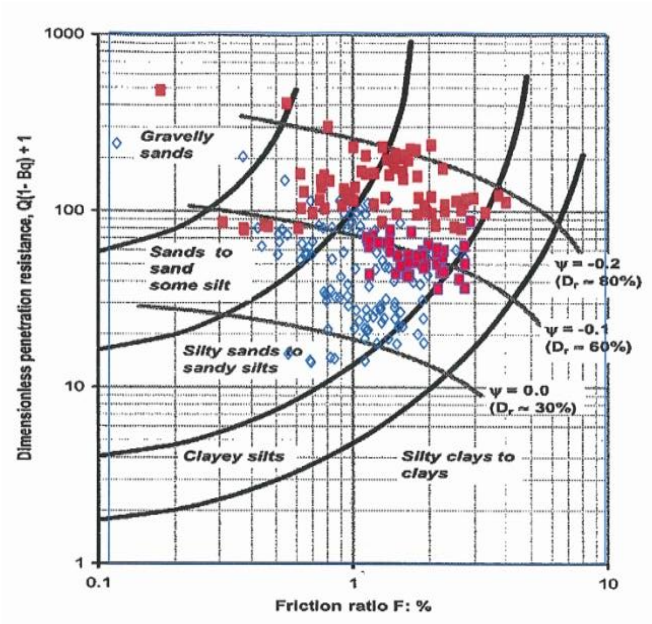

Fig. 12. *In-situ state and relative density of sandy soils*

Construction Equipment and Procedure

Limited access equipment was used for construction because of the site constraints and to minimize impact to operations during construction. Casings were driven into the ground to minimize exposure of potentially contaminated subsurface soils. The casings were driven to the top of the bedrock, encountered at depths between 42 and 58 feet. Figure 13 shows the equipment used for driving the casings for this project.

Grout mixing was done using a continuous mixer as shown in Figure 14. Grout was injected into the ground through the casing with the header and the duplex jacks for casing extraction as shown in Figure 15. The grout pump and the header were connected using a combination of high-pressure hose and rigid steel delivery lines. A pressure gage was used to measure the grout pressure to monitor the grouting process.

Quality Assurance and Quality Control Testing

Detailed quality assurance/quality control processes were implemented during grouting operations to verify that the design intent of the compaction grouting program was met. Survey was also completed to evaluate the ground movement induced by the grouting process and its impact to adjacent structures and critical utilities.

Fig. 13. Installing casings by driving using a limited access rubber track rig

Fig. 14. Mixing grout using a continuous mixer

Fig. 15. Compaction grout header and the duplex jacks for casing extraction

Extensive laboratory and field tests were also completed to evaluate the unconfined compressive strengths.

One in-situ sampling round was performed at a frequency of twice per week. Two sets of grout samples were collected per sampling round, one collected at the grout mixer and the other collected at the end of the grout delivery line. Each retrieved grout sample was used to make four test specimens. Grout test specimens from each sampling round were tested to determine the 7-day and 28-day unconfined compressive strength in accordance with AASHTO T 208.

A total of 32 compressive grout strength tests were completed for this project. The average grout strength is determined to be about 560 psi, which met the specified strength of 500 psi.

The contractor set up a laser level to monitor potential movement of the ground surface and the nearby structures during the compaction grouting work. No movement/settlement was observed on the ground surface or any adjacent structure during compaction grouting construction.

Prior to the compaction grouting, a cone penetration test (CPT) was completed near the center of the tank (CPT P-3A shown in Figure 2). Subsurface soils near the center of the tank generally consist of 26 feet of cohesive soils (i.e. clayey silt or silty clay) over inter-bedded silty sand and sandy silt to a depth of about 53 feet, where practical refusal was encountered. The tip penetration resistance of the soils encountered at the center of the tank is higher than the CPTs completed outside of the tank by a factor of more than 2; indicating that the actual soils beneath the tank have higher shear strength than the assumed values in the design.

A post-compaction grouting CPT (P-3B shown in Figure 2) was completed at the same location of the pre-grouting CPT. The post-grouting CPT showed no increase in the tip penetration resistance in the cohesive soils, and the tip penetration of the underlying silty sand and sandy silt soils increased by a factor of about 2. Practical refusal was encountered in CPT P-3B at a depth of 28 feet.

Figure 16 shows the in-situ state and relative density of the silty sand and sandy silt soils for the pre- and post-grouting conditions within the top 28 feet of CPT P-3A and P-3B. As shown in Figure 16, the relative density and in-situ state of the sandy soils explored were increased to the level assumed in the design, as presented in Figure 12 above.

HYDROTEST RESULTS

Upon completion of the new tank construction, a hydro test was completed where the tank was filled with 38 feet of water and was held for a 24-hour period. Survey was completed at 13 locations around the ring foundations when the tank was first filled with 38 feet of water and at the end of the 24-hour hold period to determine the total settlement of the tank foundations and the differential settlement between the survey locations. The survey that was completed indicated that the ring foundations settled about ¼ to ¾ inches when filled with 38 feet of water. The differential settlement between the survey locations was found to be less than ⅜ inches. The results of the survey completed during the hydro test met the required settlement limits per API-650 and API-653 Standards.

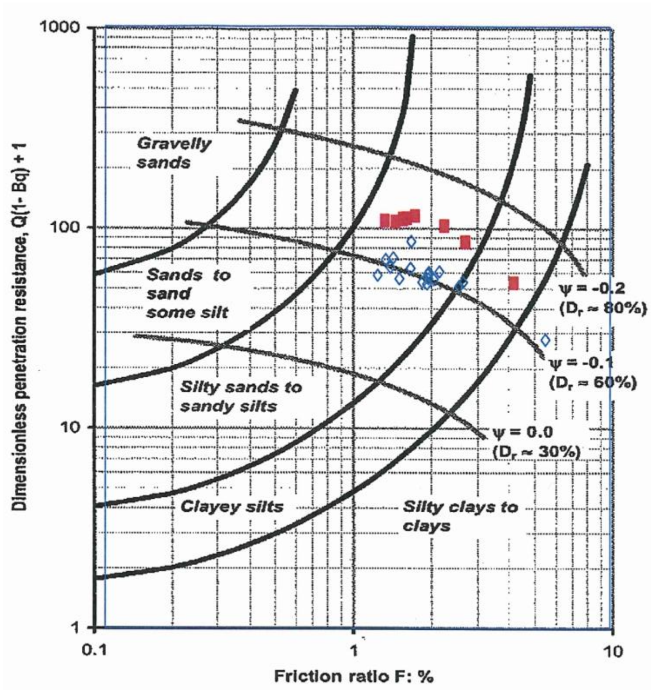

Fig. 16. In-situ state and relative density of sandy soils (Pre- and Post-grouting)

CONCLUSIONS AND RECOMMENDATIONS

As presented in this paper, compaction grouting can effectively mitigate the lateral spreading hazards induced by soil liquefaction under the design earthquake events. The estimated soil movement beneath and within the tank footprint is estimated to be small under the building code design earthquake event. Under a scenario earthquake event similar to that of the Portland Hills Fault, the estimated average soil movement beneath and within the tank footprint is computed to be less than 15 inches, greatly reduced from the estimated displacement of more than 20 feet for the pre-grouting conditions. This estimated lateral movement for the post-grouting conditions will likely cause damage to the tank but is not likely to cause the tank to collapse.

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