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EARTHQUAKE MITIGATION BY BLAST DENSIFICATION

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ABSTRACT

This paper presents a case study of blast densification of a site underlain by a loose, saturated, natural sand deposit. Densification was performed to mitigate the risk of liquefaction from earthquake-induced ground vibrations. The work was implemented as part of the design-build construction of the Marine Corps Reserve Training Center at the Westover Air Reserve Base in Chicopee, Massachusetts.

Following review of preliminary subsurface data at the site, the contractor retained a team to design and implement ground improvement by blast densification. The team, led by a geotechnical engineering firm, included a blaster, a driller, and a cone penetrometer testing firm. The team performed the analyses, design, implementation and post-densification testing to carry out and document the effectiveness of the blast densification.

The paper presents the densification program and comparison of the pre- and post blasting data including settlement results. The project duration including pre-blasting evaluation, design, implementation, and post-blasting evaluation was less than two months. This demonstrates that deep blasting can be successfully implemented as part of fast-track, design-build procurement to execute a complex ground improvement program. It also demonstrates that while the technique is not commonly used, it is sufficiently well understood to provide a flexible and cost effective alternative to the more commonly used ground improvement methods under the right conditions.

BACKGROUND

During the programming and concept design for a new Marine Corps Reserve Training Center to be constructed on the Westover Air Reserve Base in Chicopee, Massachusetts, a layer of potentially liquefiable soil was identified. The design-build contractor was tasked with densifying this layer with traditional ground improvement methods (e.g. vibro-compaction or deep dynamic compaction) to mitigate the risk of seismic-induced liquefaction at the site.

The zone requiring densification consisted of an approximately 5-meter thick layer of loose saturated clean sands at depths of approximately 6 to 11 meters within a footprint of approximately 3,700 square meters.

Because of our experience with densification blasting on a nearby site, the contractor contacted GeoDesign to discuss this method. A relatively small degree of densification was required to mitigate the earthquake-induced liquefaction risk at the site. Despite the presence of existing occupied residential buildings as close as 60m from the area requiring densification, we agreed to study the problem and design, implement, and interpret a blast densification program at the site.

The purpose of the program was to increase the density of the loose layer such that it met or exceeded the density required to resist design seismic loading (earthquake induced loading) under the Massachusetts State Building Code requirements as supplemented by published site-specific seismicity data. The design earthquake intensity used to determine susceptibility to liquefaction was based on a site-specific design magnitude (peak ground acceleration, pga = 0.082g at a range of magnitudes of 5.0M to 6.5M) (La Fosse and von Rosenvinge, 1992).

The engineering, planning and implementation of the ground improvement, included pre-improvement and post-improvement site testing. We were able to rely on previously published data and results developed by the authors during previous liquefaction studies and ground improvement of the nearby Westover Airpark North property. We also relied on published vibration results and our experience to control blast-induced vibrations to acceptable levels by means of controlled blasting techniques.

This method was considered to save time and costs because: 1) deep blasting was expected to be efficient due to the relatively low degree of densification required, and 2) because the loose (target) layer was at depth (top of the layer at a depth of 6 meters). Thus, the blast energy could be applied at the desired depth. Other ground improvement methods must expend energy to penetrate the upper dense layer to reach the deep loose stratum.

Our work included design <u>and</u> implementation of the ground improvement program as follows:

- Review of available data,
- Perform and interpret 14 pre-improvement piezocone penetration tests (uCPT's),
- Perform gradation tests on samples previously obtained in test borings,
- Design blasting and density verification testing,
- Furnish, install, and document settlement platforms,
- Perform the deep-blasting program,
- Perform seismic (vibration) monitoring during blasting,
- Analyze and interpret settlement data and modify program as needed to achieve target improvement,
- Interpret post-improvement ground settlement data,
- Perform and interpret 19 post-improvement CPT's

Based on our previous experience we decided to rely on ground settlement as the primary, and direct, method to verify the degree of densification improvement. We also compared pre-improvement CPT data to post-improvement CPT data to confirm the degree of improvement. The pre-improvement testing was also used to supplement the limited available test boring data to determine the liquefaction potential of the site soils. CPT testing was chosen over test borings because of the following advantages over cased and mudded test borings and standard penetration tests (SPTs): 1) eliminates disturbance during sampling; 2) continuous data (every 5 cm vs. every 1 to 2 meters typically); and 3) cost economies.

The post-blasting CPT's were performed two weeks after blasting to allow some time for aging of the densified soils. Aging is the phenomenon in which newly deposited or densified granular soils increase in apparent density with time, e.g. after densification as determined by SPT or CPT testing. We requested this limited period of time to allow for some aging while accommodating the project's schedule. However,

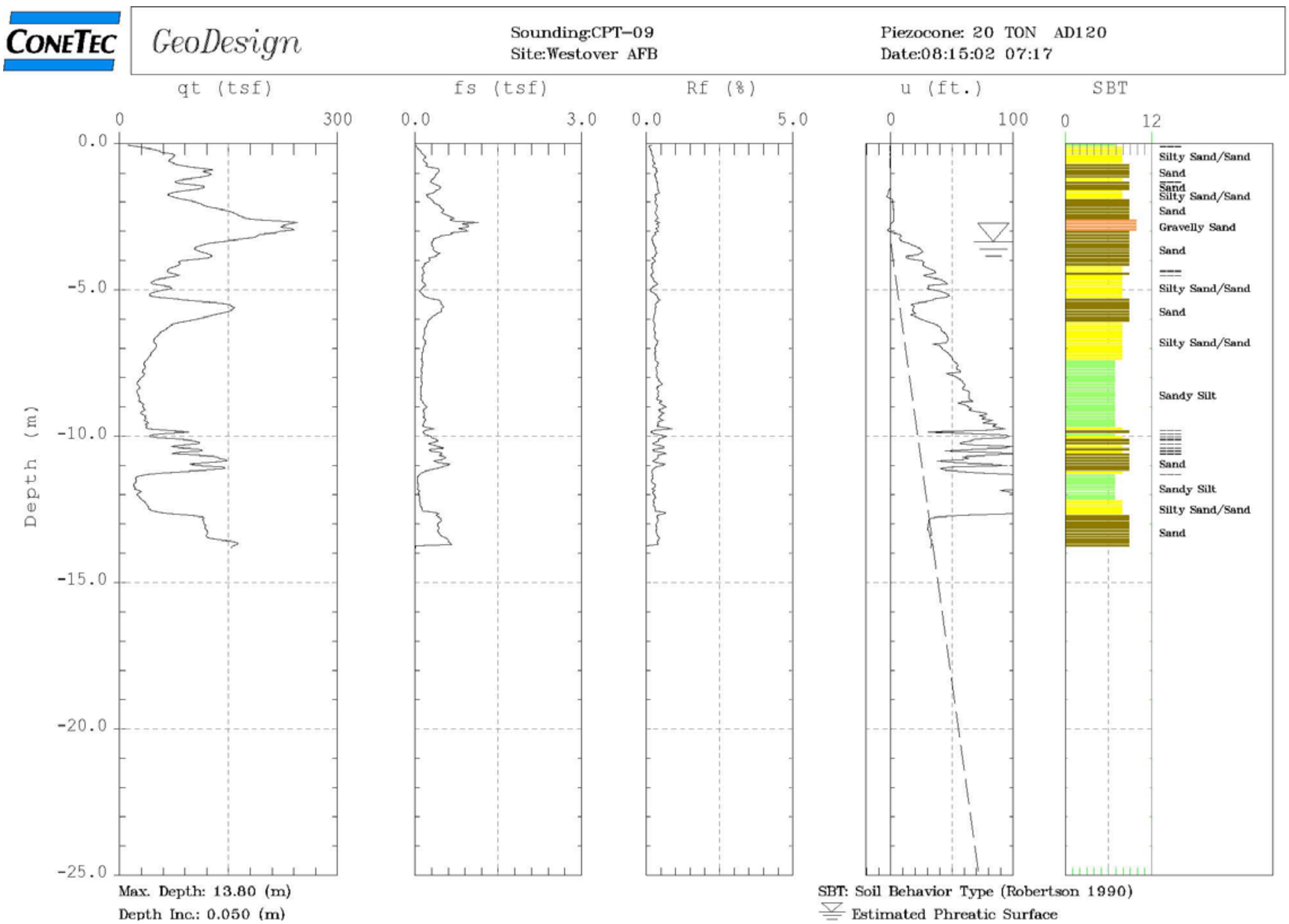

Fig. 1 – Typical Pre-Improvement CPT Sounding

based on our experience and the reports of others in the literature, we did not expect that this limited post-blasting waiting period would be sufficient to yield very high post-improvement CPT-derived strength gains. Nonetheless this data was used to correlate with the ground surface settlement in documenting improvement.

GRADATION TESTING

Nine gradation tests were performed on split-spoon samples from borings taken in the "loosest" zones as correlated by the CPT results. These tests indicate these soils consist of predominantly fine to medium sand, trace (-) silt with silt content of about 1 to 3 percent by weight. Locally the sand is fine or fine to coarse (vs. fine to medium) and/or contains up to 15 percent fine gravel. In one sample the silt content was 8 percent. This tight range of gradation indicates predominantly very clean sands.

These results confirmed that site soils, if loose and saturated, are the type most susceptible to liquefaction. The gradations also confirmed that the loose site soils were well suited to the blasting densification method.

CONE PENETRATION TESTING

To support the design of the ground improvement program we performed 14 cone penetration tests and 9 gradation tests on soils previously obtained in test borings by others. The CPT's were performed under our direction by ConeTec, Inc. on August 14 and 15th, 2002. A typical log of a pre-blasting CPT (for CPT-09) is depicted in Fig. 1.

Pre-improvement CPTs were performed in August, 2002 at 14 locations to depths of about 12 m corresponding to the maximum depths previously determined (by others) to be unacceptably loose. At three locations the CPT's were continued to 18 to 26 m depths to verify conditions at depth.

Target minimum cone tip resistance values (Qc) determined to be required to resist the 500-year return period design earthquake for this locale were used. The target minimum tip resistance was 420 lbs per square inch (psi) or 30 tons per square foot (TSF) [2.9 MPa] for an approximate 1.2 m cumulative thickness. This is roughly equivalent to a standard penetration test value of 6 blows per 30 cm. A slightly more

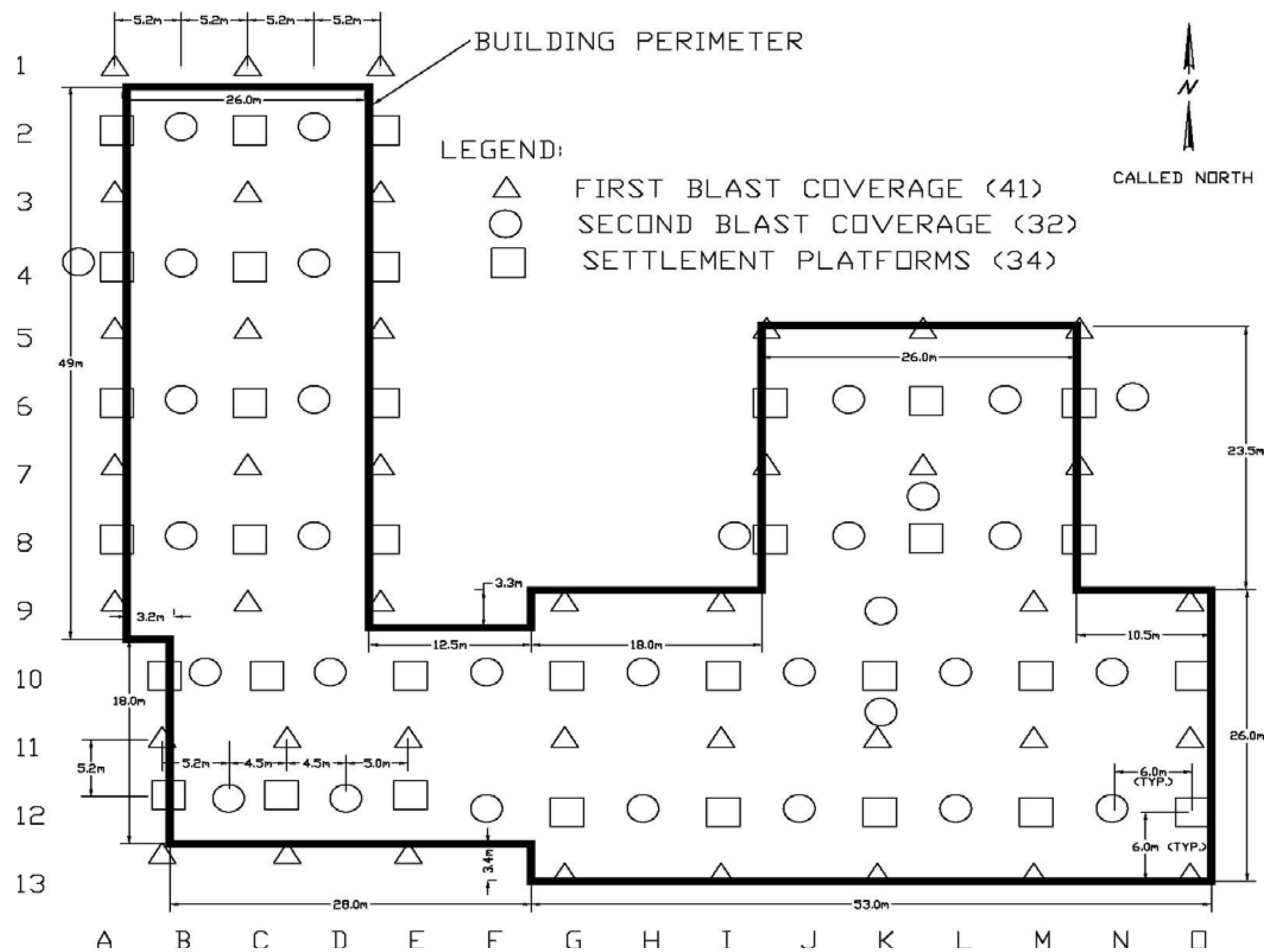

Fig. 2 - Blast Densification Plan

conservative target value of 560 psi (50 TSF) [3.9 MPa] was also considered for comparison.

The target minimum criterion was established based on a site-specific seismicity study and the Commonwealth of Massachusetts Building Code seismic design requirements. The criterion also assumed that approximately 2.5 cm of liquefaction-induced settlement could be tolerated in the event of the occurrence of the design earthquake.

Five of the 14 CPTs revealed loose, potentially liquefiable zones using the 420 psi tip resistance criterion. Using the more stringent 560 psi criterion, 12 of the 14 CPTs revealed unacceptably loose zones. Although only about a third of the CPTs indicated potentially liquefiable zones (based on the former criterion), they were not limited to one portion of the building footprint but were scattered over a large area. The liquefiable zones were sufficiently frequent and thick to indicate a meaningful liquefaction potential to require densification (ground improvement).

BLAST DENSIFICATION PROGRAM

Based on the size of the area requiring densification, the desired degree of densification, the proximity of nearby buildings, and a fast-track project schedule, we selected blast design methodology, pattern, powder factor, and charge depths that would conservatively yield the desired degree of improvement.

The blasting program consisted of a two-pass blasting sequence with overlapping square grids (Fig. 2). One week elapsed between the first and the second coverage, providing time to install the PVC pipes used to install the explosives for the second coverage and eliminating the risk of damaging pipes during the first blast coverage.

As shown on Fig. 3, two decks of high-velocity gelatin dynamite high-velocity ammonia gelatin dynamite (density 1.36 g/cc; velocity 5,500 m/sec), each about 2.7 to 3.2 kg, were detonated in a top-down sequence at each blast location. This sequence takes advantage of the pore water pressure increase following the detonation of the upper deck (7.5 to 9 m deep) prior to detonating the lower deck (10.5 to 12 m deep) and increases the efficiency of the blasting. The blasting sequence and timing delays between each hole was modified slightly based on the recorded off-site vibrations to control their magnitude to acceptable levels and protect the nearby buildings based on methods perfected and documented in La Fosse and Gelormino (1991).

In total, 41 first and 32 second coverage locations were blasted for a total of 73 x 2 = 146 exploded charges. Based on a densified thickness of 5 m and a final effective hole to hole spacing of about 7.1 m we estimate a volume of improved soil of about 18,500 m^3. In turn, this results in a final effective powder factor of 0.024 kg/ m^3 of improved ground.

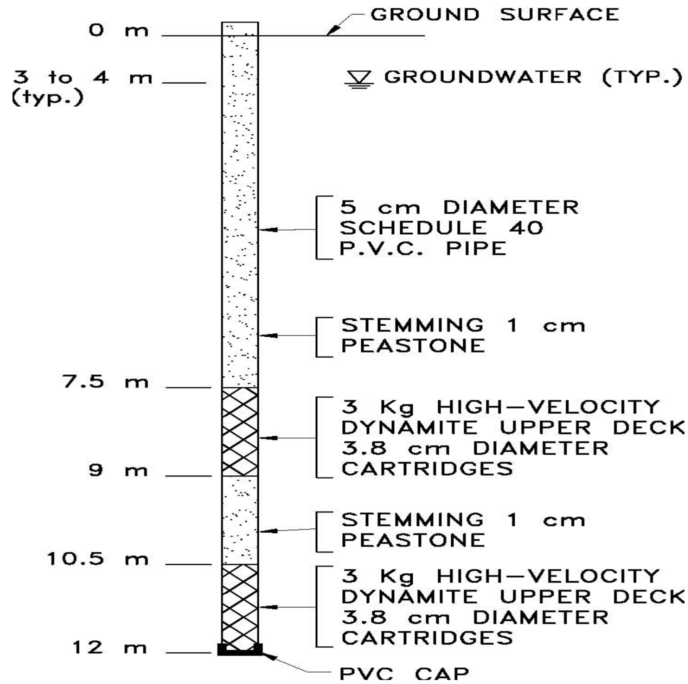

Fig.3 – Blast Densification Design Elevation

EVALUATION OF GROUND SETTLEMENT

Thirty-four shallow settlement platforms were installed at the centroid of the blast locations throughout the building footprint (Fig. 2). They were used to measure the ground surface settlement that results following densification. Their elevations were recorded by a surveyor once prior to and several times following the blasting.

The primary method of documenting the degree of densification was accomplished by comparing the settlement data and an estimated thickness of densified loose zones.

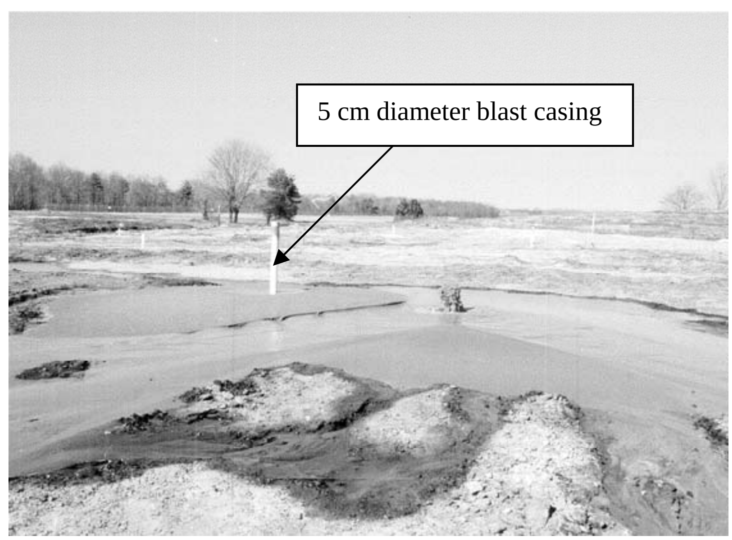

Fig. 4 – Sand Boils and Water Following Blasting

Additionally, within a few minutes following each blast, evidence of ground settlement was observed in the form of long cracks roughly encircling each area blasted in sequence, sand boils appeared, and water was discharged to the surface (Fig. 4).

Settlement was measured one day after each coverage and six days after completion of blasting. Representative ground settlement data are shown on Fig. 5.

Six days post- second-coverage blasting, the observed post-densification settlement of the ground surface varied from 8 to 31 cm with an average of slightly over 18 cm within the entire building footprint.

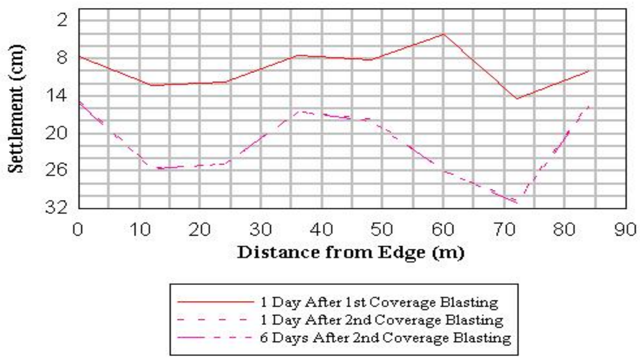

Fig. 5 – Typical Post-Improvement Settlement

Assuming the loose zone to be approximately 5 m thick on average, an 18 cm settlement equates to an average strain of almost four percent. This is an approximation as the calculation does not account for the fact that dense zones tend to dilate and may heave a small amount. Settlement was greatest within the central portions of the improved area and least at their edges. The data suggests that the loosest zones settled most and that the spatial distribution of settlement is consistent with the scatter in the pre-densification cone penetration test (CPT) data. A strain of four percent exceeds the target improvement required to resist liquefaction against the design earthquake loading. The settlement strain is about double that estimated on a nearby site that was densified by blasting in 1990 as documented in La Fosse and von Rosenvinge (1992).

COMPARISON OF PRE- AND POST-BLASTING CPT's

Nineteen post-blasting CPT's were performed on October 3rd and 4th, 2002, approximately two weeks after the second blasting coverage. The post-densification CPT's have suffixes (A or B) to differentiate them from the pre-densification CPT's.

Of the 19 CPTs taken after blasting, six (CPT-2A, 3A, 7A, 8A, 12A, and 13A) revealed localized zones of very low density (less than 30 TSF tip resistance). Some of these tests may have been taken directly over a blast hole location (which had been obliterated by site filling following blasting). Also, some of these were taken more than ten feet outside the building/blasting footprint. For these reasons, these tests were repeated within a few feet and/or moved toward the building footprint (CPT-2B, 3B, 7B, 8B, 12B and 13B). The B-series tests and the remaining 13 A-series tests generally revealed increased density in the formerly loose zones and decreased density in the formerly denser zones. The loosening was not of concern since the resulting density was still sufficiently high to resist liquefaction.

A typical log of a post-blasting CPT (for CPT-09A) is shown on Fig. 6.

Aside from the expected changes in tip and sleeve resistance of this sounding as compared to the corresponding pre-blasting sounding (Fig. 1 CPT-09/09A as discussed below), of particular note is the significant change in CPT pore water pressure after blasting. The pore water pressure was measured at the U_2 position, which is directly behind the tip and ahead of the sleeve. As seen in Fig. 4, there is relatively little excess pore water pressure generated during shearing of the soil in CPT-09A as compared to the large excess pore water pressure generated during shearing in CPT-09 (Fig. 1).

Possible explanations for the post-blasting reduction in excess pore pressures generated during CPT penetration include: 1) reduced contractive volumetric behavior of loose sands due to the densification, 2) reduced anisotropic soil permeability – possibly due to the destruction stratification of naturally occurring siltier layers, or 3) a reduction in cementation, which partially blocked drainage paths in the natural state.

Normalized tip and sleeve resistance parameters (Q_t and F_r) and the adjusted tip resistance (q_t) are affected by the pore water pressure. In addition, the SBT (predicted Soil Behavior Type that can be used for general soil classification) is also affected. Obviously, the liquefaction caused by blasting and the subsequent re-arrangement of the soil following dissipation of excess pore pressures did not change the soil type. We chose not to present the normalized data or compare q_t results; instead we simply compare the tip resistance (Qc) pre- and post-blasting as discussed below. This has the advantage that in the CPT literature Qc has been compared to relative density and the ultimate goal of this project was to increase density of loose site soils.

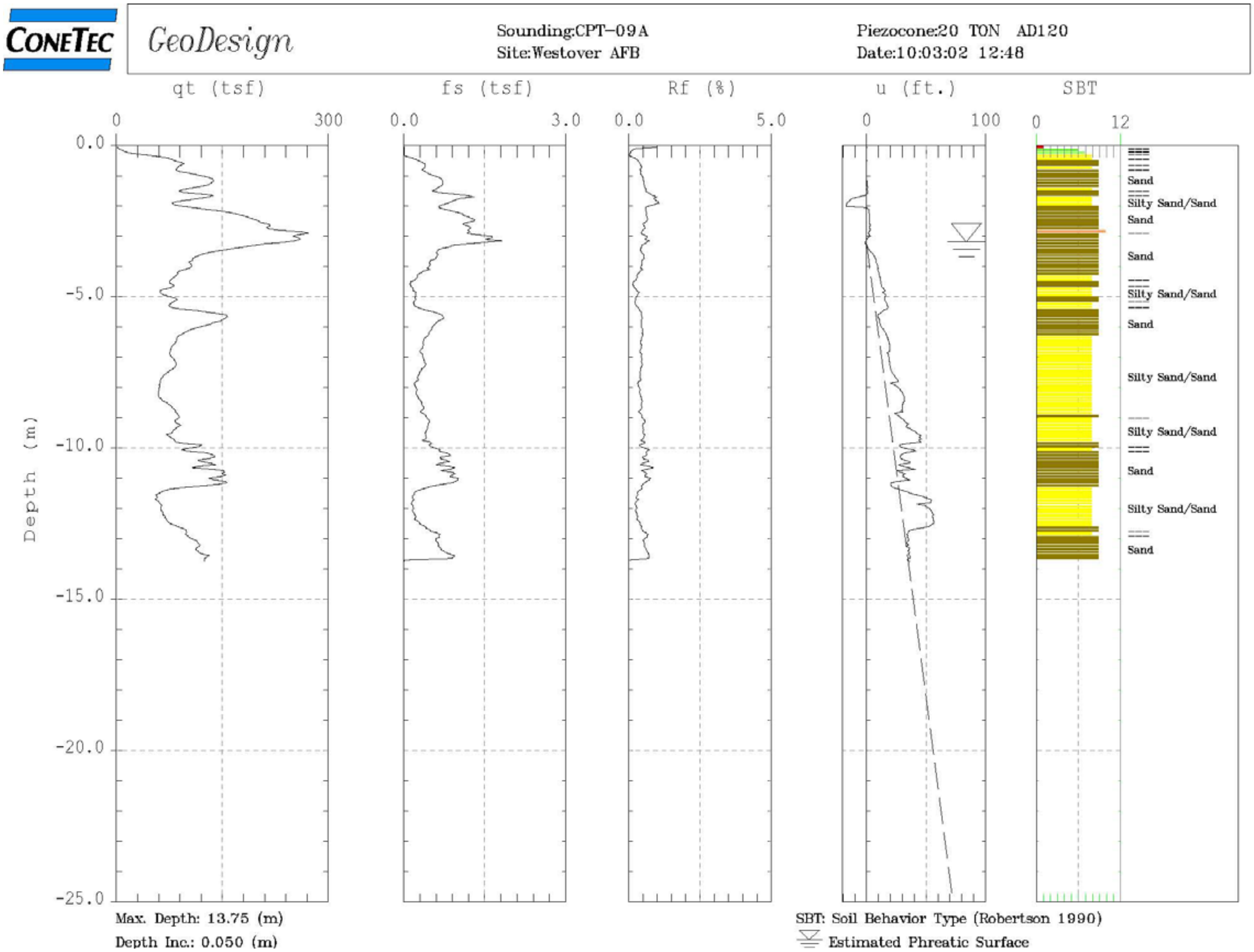

Fig. 6 – Typical Post-Improvement CPT Sounding

As noted previously, it must be remembered that when comparing CPT results for any densified sand deposit, the effects of aging must be considered.

As reported by J.H. Schmertmann, (1991), the normalized increase in static-cone bearing capacity after dynamic compaction indicates that the apparent strength of compacted silty sands increases by a factor of up to about 240% with aging. This effect is most pronounced with increasing disruption of the soil structure (e.g. 6 drops [of a 33-ton weigh dropped 105 feet] vs. 2 drops of the same weight caused more than twice the strength increase with time). If site soils behaved similarly, we would expect an approximate strength increase due to aging from the date of the post-blasting tests to final density of about 30 to 50 percent. We believe that the results which follow indicate significantly lower CPT results than can be expected long term.

Figures 7 and 8 show typical plots comparing pre- and post-blasting tip resistance (Qc) vs. depth.

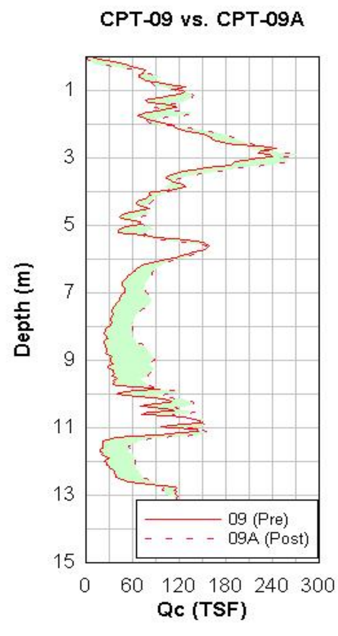

Fig. 7 – Sample Post-Improvement CPT Sounding

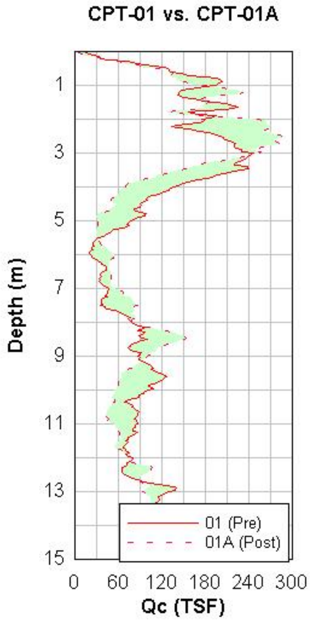

Fig. 8 – Sample Post-Improvement CPT Sounding

The increase in tip resistance in the looser zones and the decrease in the denser zones is readily apparent from these figures as highlighted by the shading. The first plot (Fig. 5) showing CPT 09/09A exhibits a greater degree of improvement that does the second one (Fig. 6) showing CPT 01/01A. The CPT 09/09A location is closer the center of the blast area (inside corner of blast area) while the 01/01A location is closer to the edge (outside corner of the blast area), which may partially explain the difference between the two improvement results. Also, the greatest ground settlement was generally observed nearer the center of the blast area.

SURFICIAL COMPACTION FOLLOWING BLASTING

Localized cones of depression, accumulated sand from "boils", and loosening of the upper site soils resulted from the blasting. These zones were readily repaired by proof compaction with conventional heavy vibratory compaction equipment. Surface compaction was documented by field density testing near or at the ground surface.

SUMMARY AND CONCLUSIONS

In summary, the blast densification program achieved the desired degree of soil improvement below the footprint of a Marine Corps Reserve Training Center. Analysis of post-densification data indicated that the previously loose and liquefaction-susceptible soils, were sufficiently densified as documented by the observed settlement and computed strain of the target layer.

Significant changes in excess pore water behavior post-blasting were also observed.

Practical lessons learned from this project include:

1. Blast densification can be successfully employed, despite the relative complexity and novelty of the method (as compared to more established ground improvement techniques) even on fast-track design-build projects.
2. This program again proved the cost effectiveness and speed of the blast densification method to improve a deep loose layer.
3. Post-blasting CPT and SPT testing may not show the final degree of improvement due to the effects of aging and insufficient waiting time common to typical construction schedules. Thus, volumetric strain estimated from ground settlement measurements should be considered the primary method to gage the results of the densification and adjust the program on a real time basis rather than CPT or SPT testing.

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