Post-production falling head testing

As a quality assurance measure, in-situ permeability testing was performed at 3 no. locations along the as-constructed cut-off wall.

At Main Dam East, 2 no. falling head tests were performed in pre-formed holes, using the method mentioned used during the pre-production test program. A third falling head test was performed in a cored hole at JG #64 as follows:

- JG064 was allowed to cure for 2 weeks.
- A 178 mm diameter open hole was drilled to a depth of 4.9 m.
- A 152 mm diameter PVC pipe was installed and the annular space between the borehole and the outside of the PVC pipe was grouted.
- Triple tube (HQ3) coring was performed to a depth extending 4 m beyond the bottom of the PVC pipe.
- A falling head test was performed using the cored section of the hole as the tested medium.

At Main Dam West, one falling head test was carried out using a formed hole and a second test using a cored hole on a production column. The third falling head test was foiled by the presence of large granular materials that were encountered in the soilcrete resulting in the PVC pipe not being able to be advanced into the fresh jet grout column; no results could be obtained.

At the Saddle Dam two falling head tests were performed in the formed holes and one test in a cored hole, as specified. The triple tube (HQ3) coring process was employed for drilling the test hole in one production column at each of the three work segments. The recovered material was delivered to the General Contractor for further inspection and testing. The results of the in-situ falling head proof tests results are listed in Table 3. All results were on the order of 10^{-6} ~ 10^{-7} cm/s and thereby satisfied the specified criterion for permeability.

Table 3. Summary of falling head proof test results

Location/Segment	Results	Remarks
Main Dam East - JG 090	9.9 x 10^{-6} cm/s	6 days curing time; formed test hole
Main Dam East - JG 113	5.7 x 10^{-7} cm/s	8 days curing time; formed hole
Main Dam West - JG 48	7 x 10^{-7} cm/s	22 days curing time; formed test hole
Main Dam West -JG 71	1.1 x 10^{-6} cm/s	9 days curing time; cored hole
Main Dam West	N.A.	Could not install pipes due to a significant amount of cobbles/boulders.
Saddle Dam - JG 043	1.2 x 10^{-6} cm/s	10 days curing time; cored hole
Saddle Dam -JG 017	4.3 x 10^{-6} cm/s	7 days curing time; formed hole

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QUALITY CONTROL PROGRAM

The quality control program included the following elements:

- All jet grouting holes were predrilled and the deviation, if any, of each hole, was measured using the SAA to verify its verticality prior to it becoming a jet grout column and to verify, by inference, that the minimum overlap was achieved. Consequently, the jet grout column locations were adjusted where necessary, or supplemental columns were added. Figure 6 shows a typical borehole deviation plan view from the SAA data.
- Grout and backflow samples were collected daily for laboratory permeability testing and unconfined compressive strength (UCS) testing. All tested samples exceeded the specified requirements for unconfined compressive strength (1.2 MPa). Table 4 provides a summary of the test results.
- Three falling head tests were performed in each segment.
- A data acquisition (DAQ) system was used to record the jet grout parameters, including the pressure, grout flow rate, rotation rate and lift rate.

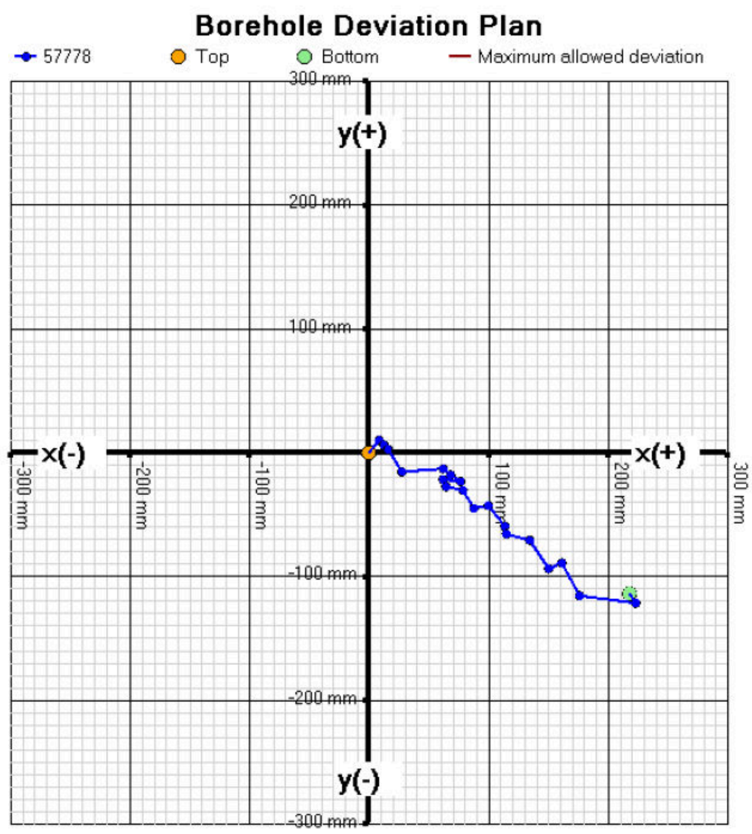

Figure 6. Typical deviation plan view from SAA data

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Table 4. Grout and backflow test results

Location	Backflow specific gravity (g/cm^3)	Grout specific gravity (g/cm^3)	Grout Marsh Time (sec)	Backflow UCS (MPa)
Main Dam East	1.70 ~1.98	1.42~1.48	36~46	3.1 to 3.7 MPa after 7 days
Main Dam West	1.72~1.92	1.45~1.48	40~46	3.3 to 4.4 MPa after 7 days
Saddle Dam	1.70~1.87	1.45~1.49	39~46	3.8 to 4.4 MPa after 7 days

LESSONS LEARNED AND CONCLUSIONS

The geological conditions at the LDI site presented numerous challenges for the construction of the jet grout cut-off wall. A comprehensive review of the site conditions and experience gained from similar projects allowed Geo-Foundations Contractors to tailor the execution of the work in order to mitigate delays, address the performance requirements and allow for contingency planning.

There are several key aspects of this project that can be adopted for work required in similar geological settings and weather conditions:

- Alignment readings at all pre-drilled holes were taken without negatively influencing the jet grout production rates and the risk of plugging jet grout nozzles.
- Pre-drilling improved productivity of jet grouting and reduced wear of jet grout drill string and tooling.
- The alignment of each jet grout hole was improved by tailoring the pre-drilling method to suit the subsurface conditions.
- The use of the SAA combined with ACI was a very effective process of verifying minimum column diameter and wall thickness.
- Forming a hole in the fresh jet grout column can be an effective, non-destructive method to check in-situ permeability of production jet grout columns.
- High concentrations of cobbles, boulders, and gravel in the jet grout treatment zone can complicate in-situ permeability testing efforts.
- Hole alignment checks at every hole using the SAA is a very effective method of confirming as-built wall geometry and allows for design adjustments, as necessary, in the field in a timely manner.

ACKNOWLEDGEMENTS

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REFERENCES

GI-ASCE. (2009). Jet Grouting Guideline. *Geo Institute of ASCE, Grouting Committee- Jet Grouting Task Force*, American Society of Civil Engineers.

Hatch. (2015). *Specification of Jet grouting for Ground Seepage Control,* Section 31 56 00.

Hatch. (2015). *Geotechnical Investigation for WMF1, H 348749-6400-229-066-0001*, dated August 21, 2015.

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Deep Power Compaction Vibro-Compaction Testing Program at Treasure Island

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Abstract

Treasure Island is located in the central San Francisco Bay, immediately north of Yerba Buena Island, between the active San Andreas and Hayward faults. Treasure Island was constructed by placing hydraulic sand fill over natural shoal deposits within perimeter rock dikes. A full-scale vibro-compaction field test, using direct power compaction (DPC) was performed to evaluate the improvement potential of sandy soils susceptible to liquefaction, and to develop a site-specific DPC vibro-compaction method specification for the desired level of densification. The test was performed at a location where the soil column consisted of approximately 22–25 ft (6.7–7.6 m) of sandy, hydraulically placed fill over 23 to 25 ft (7.0–7.6 m) of natural (Pleistocene-Holocene) shoal deposits. The DPC equipment used at the test site was configured with a vibratory hammer attached to four probes consisting of H-beams modified with steel flaps hinged to the web at the base of each beam. The test program included three intensities of compaction effort and compaction with and without prefabricated vertical drains (PVD). The test site was instrumented with vibrating-wire piezometers, wireless triaxial vibration monitors, surface settlement monuments, and reflectorless robotic total-station surveys. Cone penetration test probes were advanced before and after the DPC process to characterize the subsurface conditions and to evaluate time–dependent changes in the properties of sandy fill and shoal, or aging effects. The vibro-compaction field trials, demonstrated that (1) DPC can readily densify the sandy fill using low intensity compaction effort, (2) DPC vibro-compaction with and without PVD exhibit similar increases in post-improvement penetration resistance, and no measureable changes in time-dependent penetration resistance (aging effects), and (3) the underlying shoal deposits exhibit a different response to vibro-compaction than the sandy fill. The findings of the full-scale densification testing program were incorporated in the dynamic performance evaluation of the Treasure Island shoreline, and the geotechnical ground improvement program. The dynamic behavior of the shoal was further investigated in a separate study by ENGEO.

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INTRODUCTION

The full-scale densification testing program and corresponding results presented in this paper are part of a multi-phase geotechnical study for the Treasure Island development (TID) project. Geotechnical design considerations for the project include ground improvement techniques for liquefaction mitigation, and shoreline seismic stabilization.

As part of the TID project, liquefaction studies along with a series of full-scale densification tests were performed on loose sandy soils. The Direct Power Compaction (DPC) vibro-compaction method was used for densifying the sandy soils above and below the groundwater table. In the DPC procedure, H-piles are driven into the ground using a combination of downward and vibratory force to densify loose sandy soil. The full-scale DPC densification tests were conducted in five square cells 26.25 feet by 26.25 feet (8 meters x 8 meters) to evaluate the improvement potential of the liquefiable soils and develop a site-specific vibro-compaction method specification for the desired level of densification improvement. Two of the test cells were supplemented with pre-fabricated vertical drains (PVD). The full-scale DPC densification tests were performed in the southwest quadrant of the island. The test site selection was based on a number of factors including accessibility, distance to sensitive receptors, and the extent to which the hydraulic fill and shoal stratigraphy is typical of conditions across most of the first TID project phase.

The first step in the full-scale densification testing program was to establish fixed and variable DPC densification parameters, and determine the minimum number of test areas needed to evaluate the range of variable parameters.

A pre-test subsurface field investigation was conducted at the test site to evaluate the variation with depth of the soil density and grain size distribution of the fill and shoal deposits underlying the test site. Six CPT probes were advanced to a depth of up to 50 feet (15.2 meters), and soil samples were collected using a direct-push probe that was advanced to a depth of 51 feet (15.5 meters)

This paper contains a summary of the full-scale densification testing program, and the results of the DPC densification tests performed.

SITE CHARACTERISTICS

Treasure Island Construction. Treasure Island was constructed in 1936-37 for the Golden Gate International Exposition, with the intent to convert it to an airport following the Exposition. The island was constructed at the site of the Yerba Buena Shoals, a shallow water area directly north of Yerba Buena Island as shown on Figure 1, which depicts bathymetry from a 1926 Coastal

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Survey navigation chart. The shoals were a sand spit that extended over a mile northwest of Yerba Buena Island. The highest portions of the spit were slightly above elevation 0 (NAVD88) and it was reported to be emergent at low tide. The Bay floor areas surrounding the shoals were underlain by soft mud.

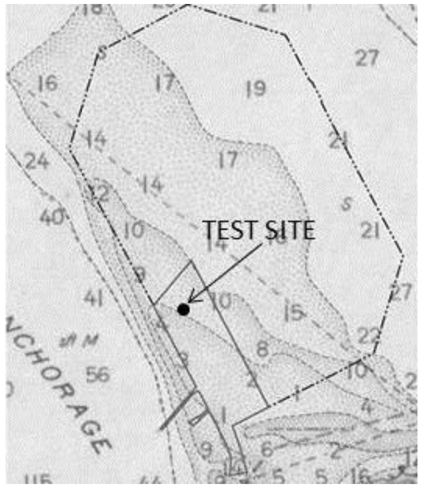

Figure 1. Coastal Survey navigation chart depicting bathymetry (1926)

Treasure Island was constructed from hydraulic sand fill dredged from Clipper Cove south of the island and from other shoals to the east. The hydraulic fills were contained by perimeter rock dikes with an initial crest elevation of approximately 14 feet above Mean Lower Low Water (MLLW).
Where the Bay floor was lower than Elevation -6 feet MLLW, hydraulic sand fill was placed up to Elevation -6 feet MLLW to support the base of the dike. Filling started at the southwest corner and progressively proceeded to the east and north. The surface of the fill was brought to an initial elevation of approximately +14 feet MLLW, but has since subsided to elevations locally as low as +6 to +8 feet MLLW due to consolidation and secondary compression of the underlying soft clay within the Young Bay Mud. Our understanding of the filling operations is based on the accounts of Lee (1969) and Hagwood (1982), and from research by ENGEO (2015).

To place the hydraulic fills, sand slurry was pumped from dredges to an on-shore distribution pipe network. Sand was discharged from nozzles, typically forming depositional cones separated by shallow pools where discharged water drained away. This method commonly results in a layered sandy fill that is well washed and poorly graded in the sandy cone areas near the nozzles, with irregularly distributed lenses of clayey sand, silt and clay in areas deposited in pools between nozzles. This depositional pattern is visible in the construction photograph, Figure 2.
The hydraulic sand fills were placed directly on the shoal soils. In many borings and CPT probes, the contact between the base of fill and top of shoal sand can be distinguished by an increase in interbedded clays and silts. The existing base of the hydraulic fills is generally a few, to as many as 20 feet (6.1 meters) lower than the pre-filling bathymetry, plotted on the cross sections.

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Across most of the island, the elevation of the base of the fill is lower than expected based on settlements measured after island construction. The difference between the elevation of the pre-filling shoal surface and the existing base of fill is likely due to several factors.
Initial settlements of the soft clays within the Young Bay Mud deposits during filling and displacement of soft sediments under dike foundations may have gone unnoticed or were simply not recorded in the available construction documentation. The greatest difference between pre-filling bathymetry and the existing base of fill occurs along the eastern shoreline where the fills are as much as 20 feet (6.1 meters) lower than the interpolated elevation of the old mud line.

Figure 2. Construction Photograph: Early stages of hydraulic filling (February 1936) showing distribution lines and sand cones emanating from nozzles in the lines as well as shallow pools between sand cones

Geotechnical Studies. Numerous geotechnical investigations, technical articles, and historic accounts of the development and geotechnical performance of Treasure Island have been prepared since the construction of the island in the late 1930s.
In 2014 and 2015, ENGEO conducted supplemental explorations for the geotechnical study of the first TID project phase. Within the onshore portions of the project, the explorations included six (6) mud-rotary borings extending to depths up to 300 feet (91.4 meters) below ground surface, and forty nine (49) CPT soundings extending to depths up to 230 feet (70.1 meters) below ground surface. A subsurface exploration for proposed offshore improvements included three (3) exploratory borings, and fifteen (15) CPT soundings.

Subsurface Conditions. The soil profile at the location of the test site is depicted in the geologic section shown in Figure 3. The soil column of the loose to medium-dense sandy soils at the field test site generally consists of approximately 20 to 25 feet (6.1 to 7.6 meters) of fill over 20 to 25 feet (6.1 to 7.6 meters) of shoal deposits. The water table is typically 7 feet (2.1 meters) below ground surface.

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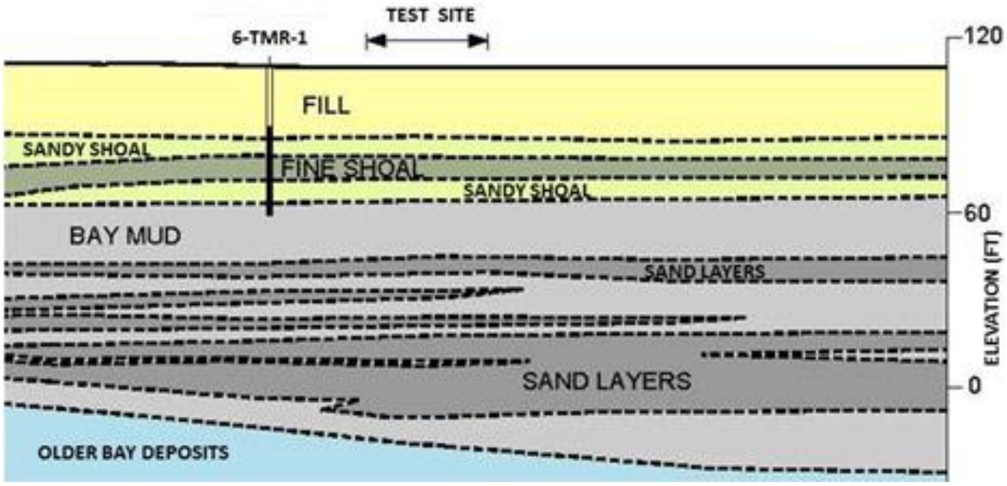

Figure 3. Test site geologic section. Elevation datum MLLW+100 feet. Horizontal scale 1inch = 100 feet

Fill Stratigraphy. The hydraulic fill consists mainly of poorly graded sand to silty sand with low fines content interbedded with thin layers of silty and clayey sand and local clay layers. The poorly graded sand layers lack any apparent cementation between grains.

Shoal Stratigraphy. At the test section location, the shoal consists of three layers that can be classified as two material types based on fines content. The upper and lower layers of the shoal typically consist of fine, moderately to well-rounded sand with approximately 10 to 20 percent fines, and exhibit relatively higher CPT tip resistance than the middle layer. The middle shoal layer typically consists of silty sand interbedded with many thin lenses of soft fat clay and clayey sand.

LIQUEFACTION POTENTIAL

Liquefaction is a phenomenon whereby saturated granular soils lose their inherent shear strength due to increased pore water pressures, which may be induced by reversing cyclic shear stresses associated with earthquakes. Low-relative-density granular soils, shallow groundwater, and long-duration and high-acceleration seismic shaking are some of the factors that cause liquefaction. Evaluation of liquefaction potential and seismically induced settlement was performed using standard-of-practice CPT-based liquefaction analysis. CPT data collected at the test site prior to the full-scale field vibro-compaction test were analyzed with the software program Cliq (Version 1.7) applying the methodologies published by Robertson (2009).

To assess the liquefaction hazard, we calculated both the Factor of Safety and the Liquefaction Potential Index (LPI) for each CPT, as defined by Iwasaki (1982). LPI is a relative hazard index, calculated on a point-by-point basis using the factor of safety against liquefaction as a function

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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.

In order to lessen the effects of post-seismic deformations, and to allow construction of new infrastructure, area-wide ground improvement in the first phase of the TID project will be accomplished using DPC vibro-compaction. This in-situ soil improvement method was chosen on the basis of technical and economic considerations, and also taking into account the existing subsoil conditions, which are characterized fundamentally by the presence of sandy soils.

FULL-SCALE VIBRO-COMPACTION FIELD TEST

A vibro-compaction field test, using DPC was performed to evaluate the improvement potential of the soils and develop a site-specific vibro-compaction method specification for the desired level of densification. The field test was performed in the southwest quadrant of the island, as shown in Figure 1, at a location where the fill and shoal stratigraphy is relatively typical of conditions across most of the first development phase. The soil column of the loose to medium-dense sandy soils at the field test location generally consists of approximately 20 to 25 feet (6.1 to 7.6 meters) of fill over 20 to 25 feet (6.1 to 7.6 meters) of shoal deposits.

The DPC method employed for the field test is a vibro-compaction technique widely used in Japan that densifies loose sandy soils by vibration and compaction. JAFEC USA was contracted to perform the DPC work. The DPC equipment used at the test site included a vibratory hammer suspended from a vibration isolation mount, which in turn was suspended from the main cable of a 270-ton crawler crane. The hammer was attached to four probes through a holder. Figure 4.

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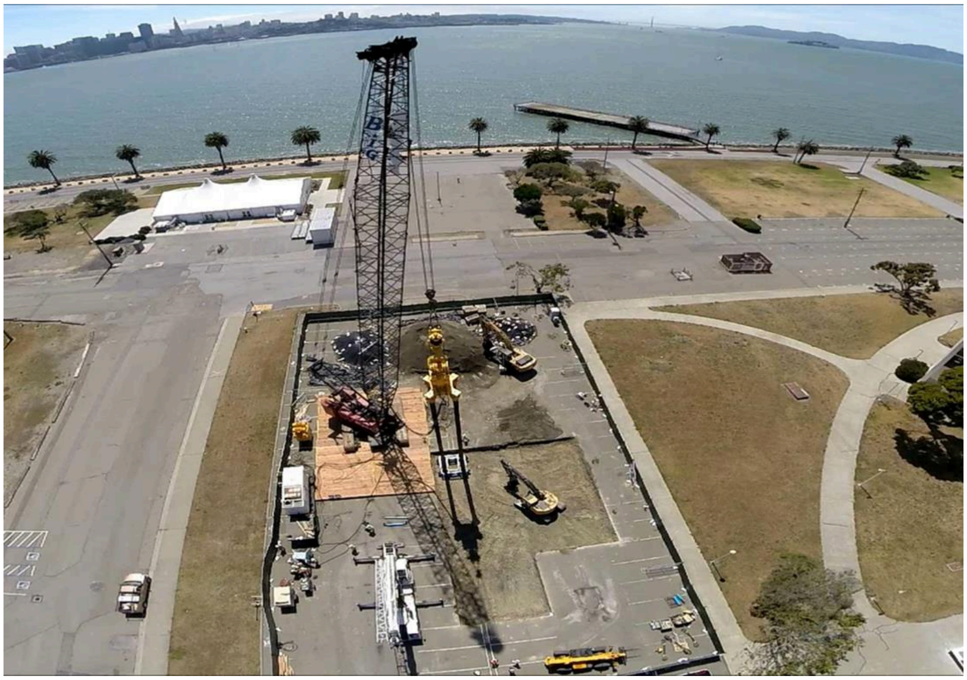

Figure 4. DPC four-probe apparatus (JAFEC USA)

The probes were H-beams modified with steel flaps hinged to the web at the base of the beam. As the beam penetrates the ground, the flaps are deployed to provide more area for compaction. During extraction of the beams, the flaps retract to reduce resistance and prevent development of voids. At the ground surface, a guide system was used to keep the beams from separating. The DPC process consisted of driving the beams to the targeted improvement depth of 45 feet (13.7 meters) below the surface. During penetration, the vibro-hammer advanced the beams to the targeted depth. As shown in Figure 5, the beams were then withdrawn incrementally a distance of U and inserted a distance of D.

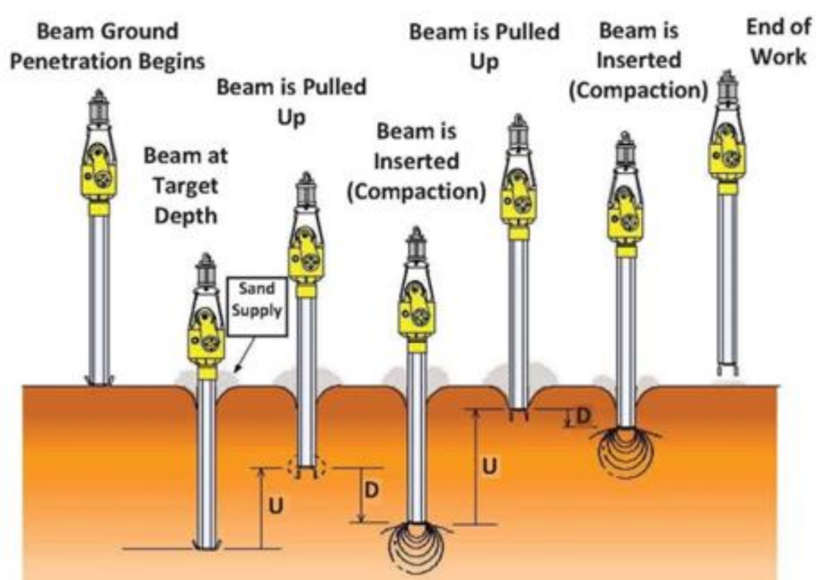

Figure 5. DPC penetration and withdrawal (JAFEC USA)

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