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ABSTRACT

This paper presents two case histories of ground improvement by dynamic compaction (DC) at the Myra Falls mine in Vancouver Island, British Columbia, Canada. Dynamic compaction was employed to densify soils at two sites within the operating mine: a waste rock dump beneath a new processing plant to reduce settlements beneath the structure foundations (Site A); and coarse fluvial and colluvial soils at the toe of an existing tailings embankment to improve seismic resistance against liquefaction (Site B).

At Site A, the variable plant loadings required variable compaction energy to achieve uniform foundation performance. At Site B, the foundation soils contained some fine grained soils that dictated a time-controlled sequential DC approach to allow excess pore pressures to dissipate between passes. Because of large uncertainties in the expected performance of DC at both sites, a fair and cost effective DC contract based on unit price per energy was adopted, instead of the traditional performance-based lump sum price contract. This paper describes the ground conditions at the two sites, DC methodologies employed, and ground improvement performance based on measurements of crater volumes and pre- and post-densification in-situ testing by Becker Penetration Tests.

INTRODUCTION

Dynamic compaction (DC) is a ground improvement method that densifies granular soils through repetitive dropping of a heavy weight (tamper) from a crane, which propagates shock waves to considerable depths. The primary goal of DC is to change a loose heterogeneous soil into one that has uniform, stronger engineering properties. In materials containing saturated fine-grained soils, the compaction process is complicated by the creation of excess porewater pressure during compaction, a phenomenon which reduces the effectiveness of the subsequent compactive passes unless the excess pore pressures are adequately dissipated. Generally, DC is limited to soils with less than about 15% passing the No. 200 sieve. Dewatering may also be required to lower the groundwater table before the required ground improvement can be achieved.

This paper presents two case histories of ground improvement by dynamic compaction at the Myra Falls mine in Vancouver Island, British Columbia, Canada. Dynamic compaction was employed to densify soils at two sites within the operating mine: a waste rock dump beneath a new processing plant to reduce settlements beneath the structure foundations (Site A); and coarse fluvial and colluvial soils at the toe of an existing tailings embankment to improve seismic resistance against liquefaction (Site B).

At Site A, the variable plant loadings required variable compaction energy to achieve uniform foundation performance. At Site B, the foundation soils contained some fine grained soils that dictated a time-controlled sequential DC approach to allow excess pore pressures to dissipate between passes. Because of large uncertainties in the expected performance of DC at both sites, a fair and cost effective DC contract based on unit price per energy was adopted, instead of the traditional performance-based lump sum price contract. This paper describes the ground conditions at the two sites, DC methodologies employed, and ground improvement performance based on measurements of crater volumes and pre- and post-densification in situ testing by Becker Penetration Tests.

SITE A

Site A Ground Conditions

Site A comprises a rock dump constructed on the side slope of a valley. Bedrock is exposed on the natural slope immediately to the north of the site and dips steeply to the south. The thickness of the loose-dumped waste rock varies from 5 m on the north side to more than 26 m on the south side. The waste rock is underlain by a relatively thin layer of colluvium overlying bedrock. The water table follows the contact of the waste rock and colluvium. A tailings management facility was actively depositing tailings against the lower slope of the waste dump. Fig. 1 shows a typical geologic section through Site A, including the DC treatment area for the new processing plant.

The waste rock fill consists of highly variable mixtures of silt, sand, gravel, cobbles and boulders up to about 0.6 m size. The fill was also observed to contain tree trunks up to 0.3 m diameter and other general mine refuse. The underlying colluvium comprises mostly sand and gravel. Becker Penetration Tests (BPT) during site investigation indicated loose zones, typically 1 m to 3 m thick, present throughout the waste rock fill and colluvium.

Plant Foundation Considerations

The processing plant included a 33 m long by 22 m wide plant building, a 25 m diameter thickener, and a 12 m diameter water tank. The key foundation design considerations are settlement of the dumped waste rock due to the additional imposed structure loads, and adequate setback from the crest of the slope to provide stable foundation support.

The imposed structure loads of up to about 200 kPa bearing pressure would result in unacceptably large total and differential settlements of the plant foundations if the existing ground was left untreated. It was therefore decided to improve the upper 8 m to 10 m of the rock fill to reduce potential settlements of the plant foundations.

The process plant was set back 15 m from the crest of the 40 m high rock fill slope, which has a slope of 20°, or approximately 2.5H to 1V. Slope stability analyses indicated that this setback distance will provide adequate static factor of safety for the plant. Post-earthquake slope stability analyses, which assumed liquefaction of the saturated colluvium, also indicated that the waste dump will be stable, but could undergo some permanent ground displacements due to the design earthquake.

Site A Ground Improvement Methodology

The process plant footprint was classified into two areas on the basis of structure loading or foundation bearing pressure: Area 1 under the large and heavy thickener structure, and Area 2 under the balance of the plant. To achieve uniform plant foundation performance, the DC ground improvement design comprised “high energy” in Area 1 to achieve treatment to 10 m depth, and “low energy” in Area 2 to treat to 8 m depth. In addition to the dual energy approach and because of the highly variable rock fill, a staged observational approach to checking ground improvement by in-situ testing was adopted. Phases 1 and 2 of the 3-phase DC program were carried out, then interim BPTs were performed to check ground improvement; the program was adjusted as needed, before proceeding with Phase 3 and final BPTs.

A 125 ton BLH Lima 1200 SC crawler crane modified for DC with a 33.5 m long boom was used to lift and drop a 15.9 tonne (17.5 ton) steel tamper with a single cable. The drop height was 24 m to deliver a potential impact energy of 381.6 tonne-m (i.e. 15.9 tonne x 24 m) per drop at all compaction points. The tamper was octagonal shaped and measured 1.8 m across the sides (flats).

Figure 2 shows the DC treatment layout at Site A, which covered an area of approximately 37 m by 91 m in plan. The treatment pattern extended approximately 6 m outside the foundation footprint. The total program consisted of 133 treatment points at 5.25 m grid spacing over the treatment area. The DC work was carried out in three phases of tamping, as shown in Fig. 2. The spacing between tamping points for Phase 1 and 2 was 10.5 m, with lesser spacing between the Phase 3 points.

Table 1 summarizes the DC work, including the energy applied for each phase. As shown, the more heavily loaded Area 1 received twice as many drops per point (energy per treatment point) as those in Area 2.

The total impact energy delivered to the entire treatment area was 1,270,728 tonne-m. The applied energy per unit volume of treated soil was 52.8 tonne-m/m^3 for Area 1, assuming a treatment depth of 10 m, and 35.6 tonne-m/m^3 for Area 2, assuming 8 m treatment depth.

An “ironing” pass to densify the upper 2 m of the ground was completed at the end of the DC program. The ironing pass consisted of two drops from 18 m height at the Phase 1 and 2 points, and 1 drop from 12 m elsewhere so that the tamper footprints generally touched. Energy from the ironing pass is not included in the values reported in Table 1.

Site A Ground Improvement Performance

Vibration monitoring was conducted during DC using a seismograph and geophone arrangement. The geophone was placed on the ground surface at various distances from the treatment point and weighed down with a bag of soil. Recorded peak particle velocity values at various distances are shown on Fig. 3. Vibrations induced by DC decreased rapidly with distances beyond 10 m.

Craters formed at each tamping point were backfilled with imported rockfill at the completion of each phase. Crater volumes were estimated based on the measured dimensions of the top diameter and depth for each crater, and a truncated cone shape. The average crater volumes per treatment point are summarized in Table 1. As expected, the volume of the craters decreased from Phases 1 to 3 as the energy applied by each phase reduced and the density of the foundation increased from previous drops. Several “tamper tests” were completed at select locations where the crater was measured after every 5 drops. These “tamper tests” provided valuable information on the degree of compaction that can be achieved with increasing number of tamper drops. Fig. 4 shows the measured cumulative crater volume versus the number of tamper drops for a typical tamper test in Area 1. The plot shows diminishing increase in crater volume after about 55 to 60 drops.

The total estimated volume of craters was 708 m^3 in Area 1 and 709 m^3 in Area 2. This implies that about 5.1% of ground settlement was induced in Area 1, and 4.6% in Area 2, assuming a 10 m treatment depth for Area 1 and 8 m for Area 2. On average, the ground surface over the treatment area is inferred to have settled 0.51 m and 0.37 m in Areas 1 and 2, respectively.

To confirm the effectiveness of DC in reducing settlements, Becker Penetration Test (BPT) were completed at the site prior to the DC program (twelve), at the completion of Phase 2 (four), and at the completion of Phase 3 (six). The Becker hammer drill uses a double-acting diesel pile hammer to drive a double-walled casing into the ground. When completing a BPT, the casing is driven close-ended, in which case blow counts necessary to drive the casing 300 mm are recorded. For drilling and sampling, the casing is driven open-ended, in which case cuttings are forced up the inner pipe to ground surface to provide continuous soil sampling. At most test locations, side by side BPT and open-end casing sampling holes were carried out.

Fig. 5 shows the average BPT blow count profiles for pre-DC, after Phase 2 DC, and after Phase 3 DC. Note that the blow count shown, $(N_b)_{30}$, is the measured blow count normalized to 30% of the Becker hammer energy as proposed by Sy and Campanella (1993). Comparisons of the BPT blow counts indicate the following:

- At the completion of Phase 2, there was a general increase in BPT blow counts in the loose zones between 5 m to 10 m depth of 10 to 20 blows/300 mm in Area 1 and 5 to 10 blows/300 mm in Area 2.
- At the completion of the DC program, there was a general increase in BPT blow counts relative to pre-DC of 10 to 30 blows/300 mm to a depth of 8 m to 10 m.
- No significant improvement occurred below the target compaction depth of 8 m to10 m.

Settlement analyses for the post-DC foundation indicated up to an 80% reduction in the anticipated “pre-DC” foundation settlements, with an average reduction of 60% in the key areas of the plant foundation.

SITE B

Site B Ground Conditions

Site B is located at the toe of an existing 20 m high tailings impoundment that stretched about 1000 m long adjacent to a creek. In the eastern 600 m of the site, loose to dense fluvial gravels and sands, with discontinuous thin zones of silts, clays and organic materials are present. Beneath the fluvial deposits are interbedded layers of uniformly graded sands and fine sands and silts; and lacustrine deposits of silts and clays sit above bedrock. In the western 400 m of the site, a continuous zone of loose, silty gravel colluvium ranging in thickness between 5 m and 8 m overlies the fluvial deposits. The water table was generally within about 1 m of the ground surface and controlled by the adjacent river. Fig. 6 shows a generalized geologic section (looking north) along the length of Site B.

Berm Design Considerations

Site investigations and seismic analyses indicated widespread liquefaction of the existing tailings would occur due to the design earthquake, leading to breach of the tailings impoundment. Accordingly, a stabilizing berm consisting of compacted select rock fill was designed to buttress the existing tailings dam.

Liquefaction assessment of the foundation soils also indicated liquefaction of the shallow colluvium in the west half of the site, and local layers and pockets in the upper 8 m of the fluvial deposits in the east half of the site represents the greatest threat to the stability of the proposed stabilizing berm. Liquefaction and strength loss in these units would cause a sliding failure of the stabilising berm founded on the liquefied soil, or result in horizontal berm displacements which would exceed the allowable design criteria. Therefore, densification of the upper 8 m to 10 m of foundation soils by DC was required beneath the base of the new berm to increase the sliding resistance of the berm and to control dynamic displacements. Fig. 7 shows a cross-section of the proposed seismic upgrade berm and underlying DC treatment zone.

Site B Ground Improvement Methodology

The objectives of the DC program at Site B were to increase the density of the foundation soils so that they respond in a dilative manner during undrained shearing (as opposed to contractive response for liquefiable soils) and have a high average frictional resistance. The performance specifications for the DC, based on Standard penetration tests (SPT), were as follows:

1. The average $(N_1)_{60\text{-}cs}$ for the soils in the upper 10 m should be greater than 25 to prevent liquefaction and to maximize the frictional resistance of the densified foundation soils.
2. The target minimum $(N_1)_{60\text{-}cs}$ is 20.
3. Local pockets of soil that do not achieve $(N_1)_{60\text{-}cs} \geq 20$ may exist, but they should not represent more than 10% of the total densified foundation and not be continuous. For these local pockets, a minimum $(N_1)_{60\text{-}cs}$ of 15 was specified.

Note that $(N_1)_{60\text{-}cs}$ refers to SPT blow count normalized to 100 kPa effective overburden pressure and 60% of the potential energy of the SPT, and corrected to equivalent clean sand as proposed by Youd et al (2001). Because the predominantly coarse grained soils limited the applicability of the SPT, BPTs were used to estimate equivalent SPT $(N_1)_{60}$ values following the BPT-SPT interpretation procedure proposed by Sy (1993). The Sy method requires measurement of Becker hammer transferred energy and casing friction.

The Site B treatment area comprised a 1000 m long by 15 m to 30 m wide zone along the toe of the tailings impoundment. The site was divided into two zones: the western 400 m long Zone I underlain by the silty gravel colluvium, and the eastern 600 m long Zone II underlain by sand and gravel fluvial deposits with intermittent fine-grained soil zones or pockets.

The same BLH Lima crawler crane and tamper used at Site A were also used at Site B. The design drop height was 24 m in Zone I and 23 m in Zone II to deliver a potential energy of 381.6 and 365.7 tonne-m per drop, respectively. Two metres of rockfill were placed at Site B to construct a level platform to maintain the ground surface about 2 m to 3 m above the water table for conducting the DC.

The DC program included 635 treatment points broken into two and three phases as summarized in Table 2. The target densification depth was 10 m in Zone I and 8 m in Zone II. The width of the densification was about 20 m to 25 m in Zone I and about 16 m in Zone II.

To overcome excess pore pressures developing in the silty zones of the foundation during DC that would limit the effectiveness of further drops, a time-delayed multiple-pass phase approach was adopted. Phase 1 and Phase 2 drops in Zone I were completed in 3 passes of 20 drops each to allow for dissipation of pore pressures between passes. Phase 1 drops for Zone II were completed in 2 passes of 20 and 15 drops. Pneumatic piezometers were installed in the silty foundation zones within the treatment area, generally between treatment point locations, to measure pore pressures during DC. The objective was to ensure excess pore pressures had dissipated by at least 75% before the subsequent DC pass was conducted.

The total impact energy delivered to the Zone I area was 4,218,588 tonne-m. The DC area of Zone I was 8,164 m^2, and assuming a treatment depth of 10 m, the applied energy per unit volume of treated soil was 51.7 tonne-m/m^3. The total impact energy delivered to the Zone II area was 3,593,002 tonne-m. The DC area of Zone II was 7,480 m^2, and assuming a treatment depth of 8 m, the applied energy per unit volume of treated soil was 60.0 tonne-m/m^3.

Site B Ground Improvement Performance

Fig. 8 shows a photo of the DC and associated craters at Site B. Similar to Site A, crater volumes were measured based on the dimensions of the crater top diameter and depth. The total estimated crater volume was 3,428 m^3 in Zone I, and 2,715 m^3 in Zone II. This implies that about 4.2% of ground settlement was induced in Zone I, assuming a 10 m treatment depth, and 4.5% in Zone II, assuming an 8 m treatment depth. On average, the ground surface over the area of compaction is inferred to have settled 0.42 m in Zone I and 0.38 m in Zone II.

Readings of the foundation piezometers were collected twice each day when DC was in the vicinity of the piezometers, and less often otherwise. Several tests were conducted where pore pressure response was measured every five blows at a drop point adjacent to the piezometer cluster.

In general, pore pressures increased due to the applied DC energy but were found to dissipate quickly such that excess pore pressures were completely dissipated prior to the next pass. Pore pressures at locations with silty soils displayed a more dramatic response to the DC energy, but again dissipated very quickly. Fig. 9 is a plot of typical pore pressures recorded at a piezometer cluster during DC at adjacent treatment points. The shallow piezometer at 5.5 m depth showed pore pressure response due to 3 passes of DC at an adjacent Phase 1 treatment point and at an adjacent Phase 2 treatment point. The deeper piezometer at 8.5 m depth showed minimal response, likely because the piezometer tip was not embedded in a silty layer capable of generating large excess pore pressures.

The effectiveness of the DC was verified through post-DC BPTs. BPT blow counts were interpreted to equivalent SPT $(N_1)_{60}$ values using the Sy (1993) method. A pile driving analyzer was used to measure the transferred energy of the Becker hammer. Casing friction was measured in the field with a load cell after each casing break or add-on. Fig. 10 shows results of one post-DC BPT.

Figure 11 shows typical pre-DC and post-DC equivalent $(N_1)_{60}$ profiles interpreted from nearby BPTs. Overall increases in $(N_1)_{60}$ were obtained in the looser soil zones. As would be expected, denser soils with $(N_1)_{60} > 30$ showed little or no increase. Post-DC testing verified that, on average, $(N_1)_{60}$ greater than 25 was achieved in both Zones I and II.

Local zones of $(N_1)_{60}$ less than 20 were encountered, but these zones did not appear to be continuous between adjacent test locations. Open-casing sampling holes showed that some of these zones were localized pockets or layers of higher fines content material, typically only 0.3 m to 0.6 m thick and surrounded by coarse-grained soils that would allow rapid drainage of pore pressures generated by earthquake shaking.

Casing friction measured in pull-up tests during the post-DC BPTs showed a significant increase compared to measurements from the pre-DC tests, as shown in Fig. 12. The increase in casing friction through the upper 10 m of foundation soils is further confirmation of the foundation improvement achieved by DC. Similar trends from other compacted sites have been noted in Sy (1997).

CONCLUSIONS

Dynamic compaction was shown to be an effective means of ground improvement for two project sites underlain by variable coarse grained soils and to achieve different objectives, i.e. densification to reduce total and differential settlements at Site A; and densification to increase frictional resistance and liquefaction resistance at Site B. The DC was shown to have a depth of influence up to 10 m and was able to induce 4% to 5% average ground settlement over the treatment area. The DC improved ground satisfactorily met the design objectives at both sites. The effective use of Becker Penetration Tests to monitor performance of ground improvement on rockfill and gravelly soil sites was illustrated by the two case histories presented.

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Table 1 - Dynamic Compaction Summary at Site A

Treatment Area	Phase	No. of TP	No. of Drops Per TP	Average Crater Volume Per TP (m^3)	Total Input Energy (tonne-m)
1 ($1,373\ m^2$)	1	13	60	18.6	328,176
	2	13	40	16.0	198,432
	3	26	20	8.3	198,432
				Total Applied Energy	725,040
2 ($1,917\ m^2$)	1	21	30	14.4	240,408
	2	20	20	11.1	152,640
	3	40	10	5.0	152,640
				Total Applied Energy	545,688

Note: TP = Treatment point.

Table 2 - Dynamic Compaction Summary at Site B

Treatment Area	Phase	Grid Spacing (m)	No. of TP	No. of Drops Per TP	No. of Passes Per TP	Average Crater Volume (m^3)	Total Input Energy (tonne-m)
Zone I (Sta. 0+000 to 0+400)	1	11 x 12	81	60	3 (20 ea.)	19.5	1,854,576
	2	11 x 12	66	60	3 (20 ea.)	16.2	1,511,136
	3	11 x 12	149	15	1 (15)	5.2	852,876
Zone II (Sta. 0+400 to 0+1000)	1	7 x 8	203	35	2 (20,15)	9.5	2,598,298
	2	7 x 8	136	20	1 (20)	6.2	994,704

Note: TP = Treatment point.

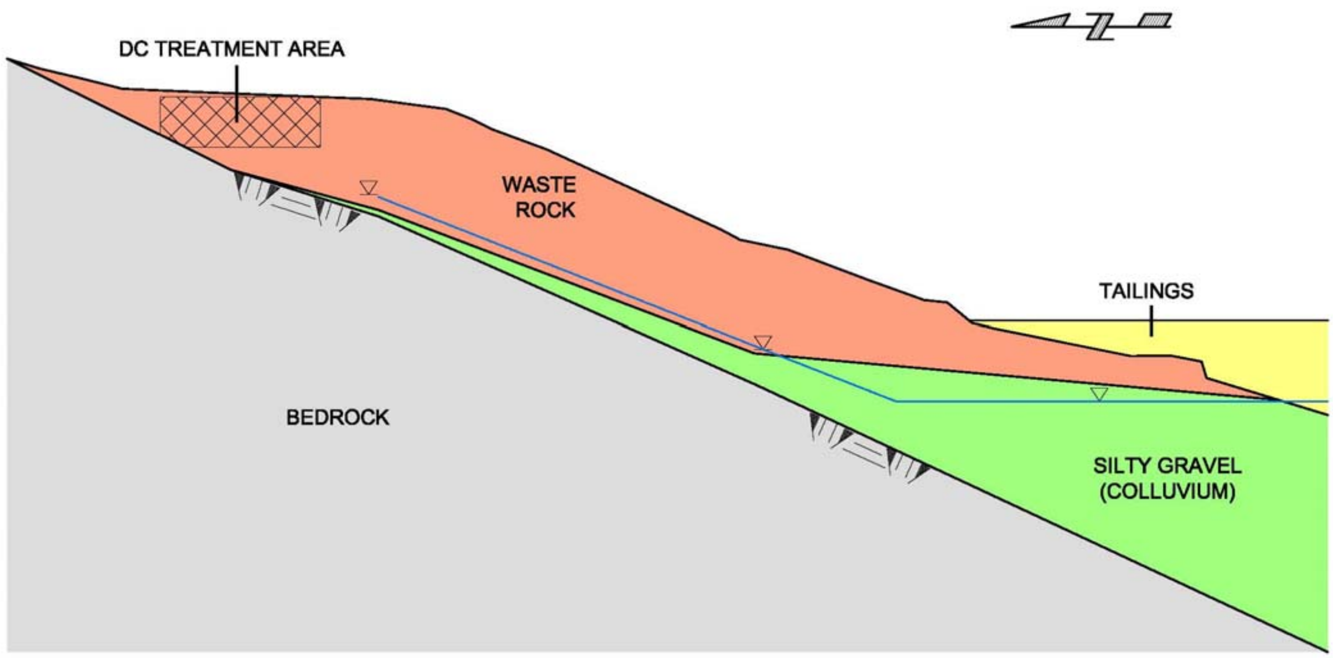


Fig. 1 - Geologic Cross-Section at Site A

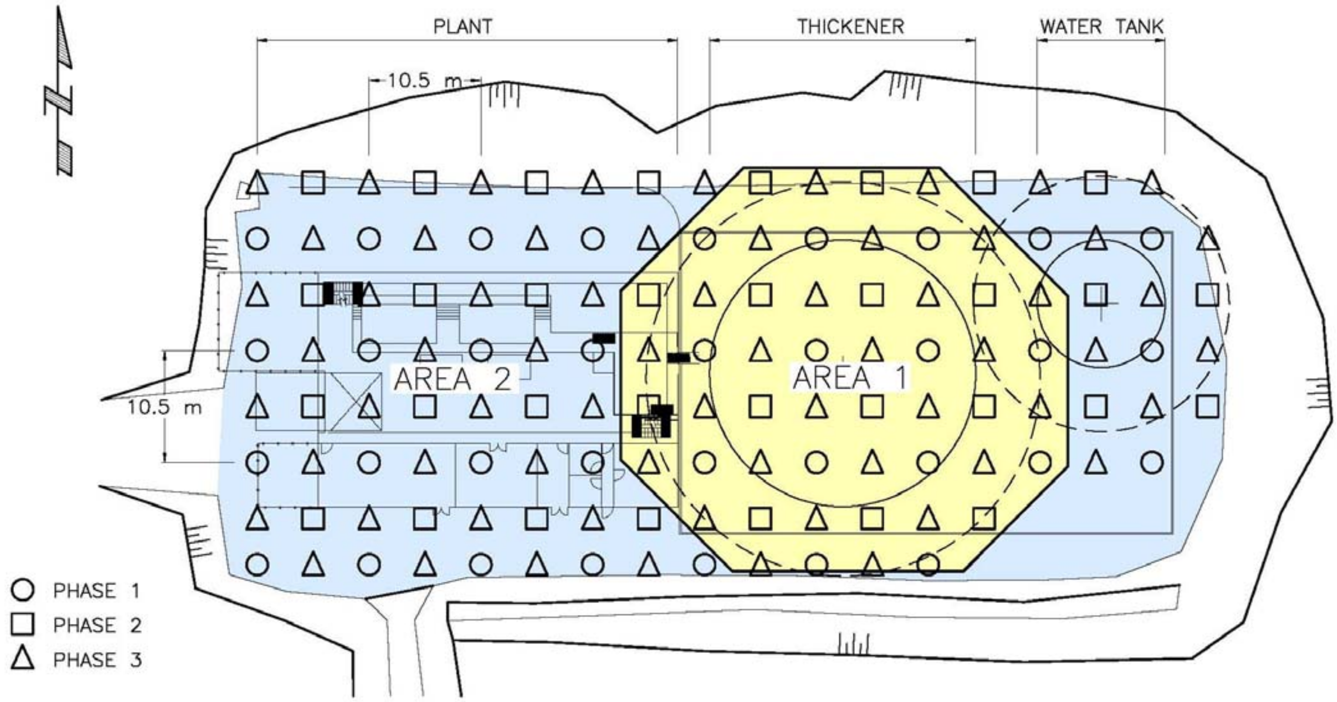


Fig. 2 – Dynamic Compaction Layout at Site A

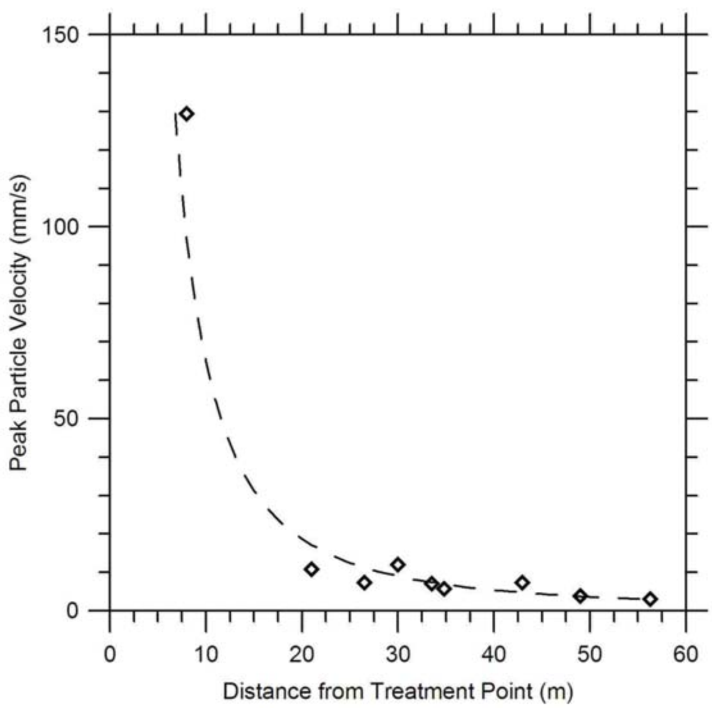


Fig. 3 – Vibration Measurements at Site A

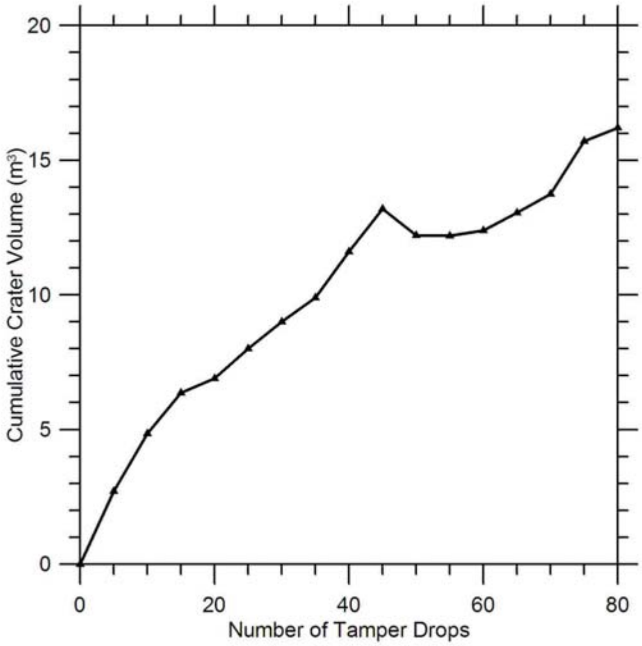


Fig. 4 – Crater Volume vs. Number of Tamper Drops at Site A

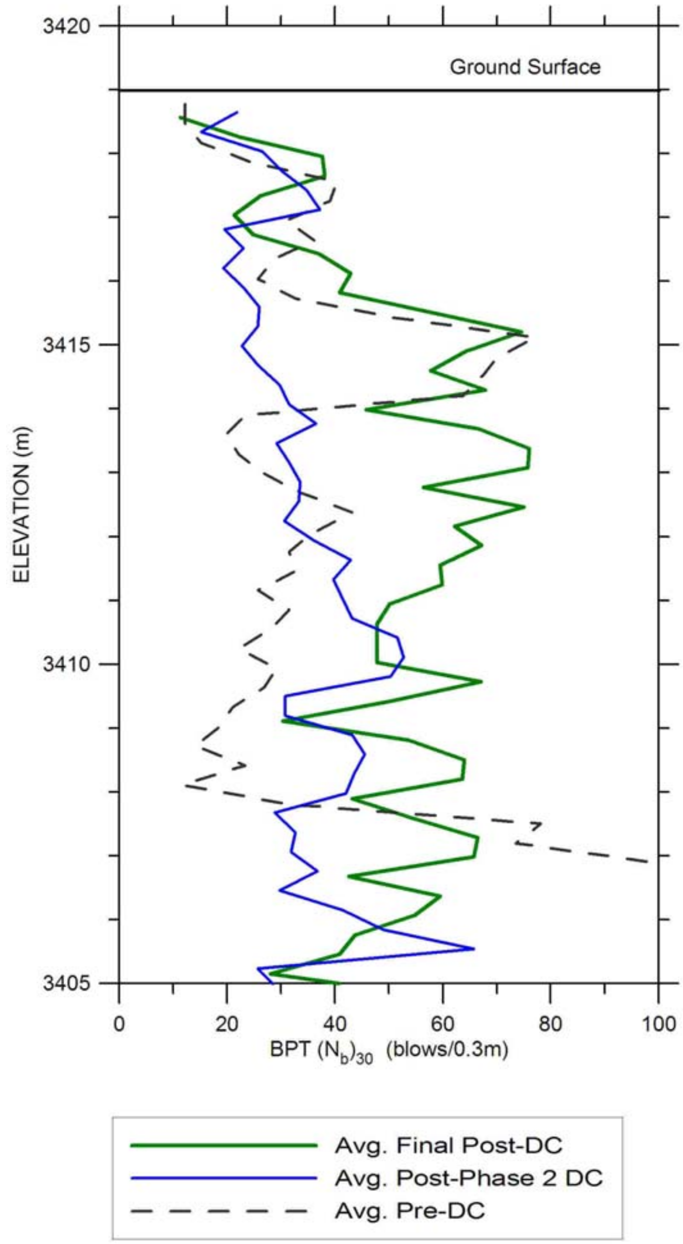


Fig. 5 – Pre-DC and Post-DC BPT profiles at Site A

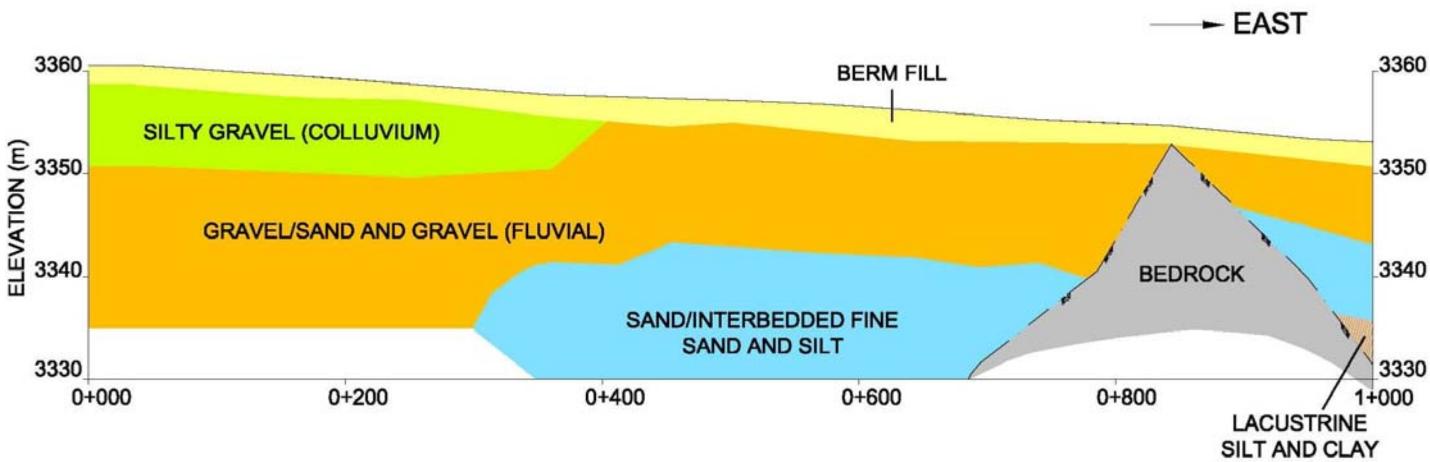


Fig. 6 – Geologic Section at Site B

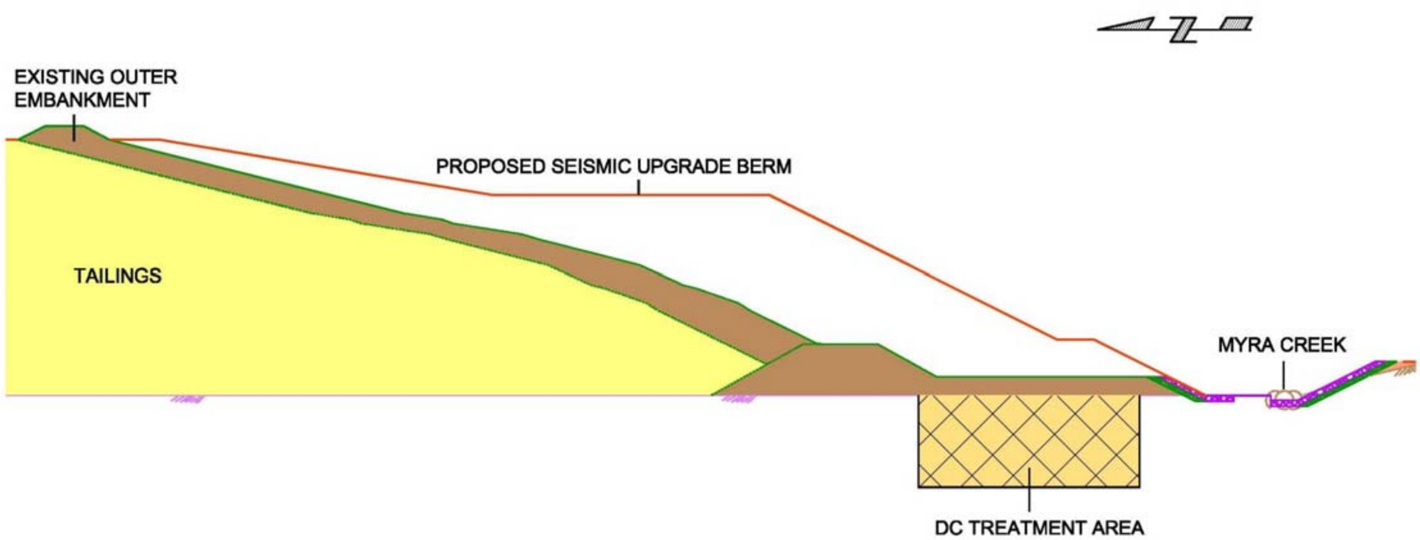


Fig. 7 – Cross-Section of Seismic Upgrade Berm at Site B

Fig. 8 – Photo of DC and Craters at Site B

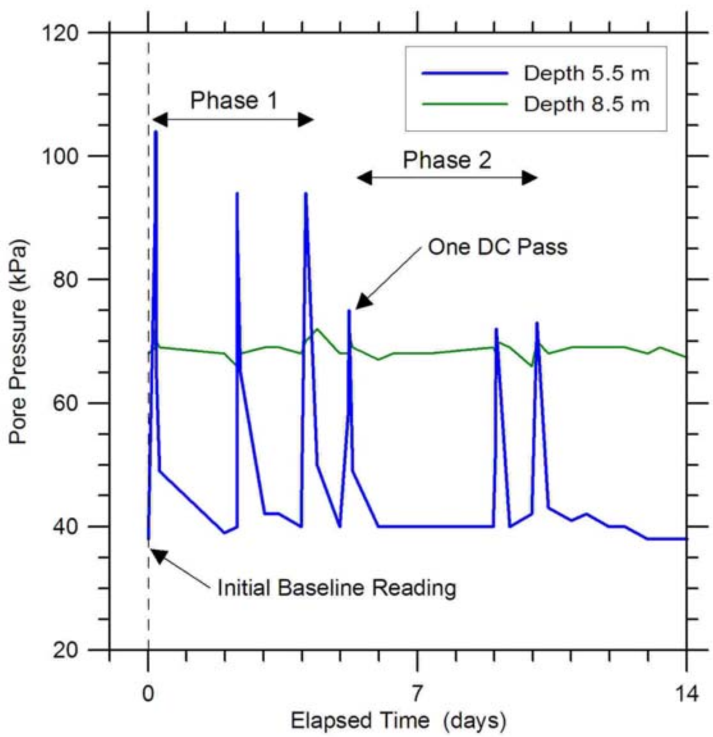


Fig. 9 – Typical Piezometer Response at Site B

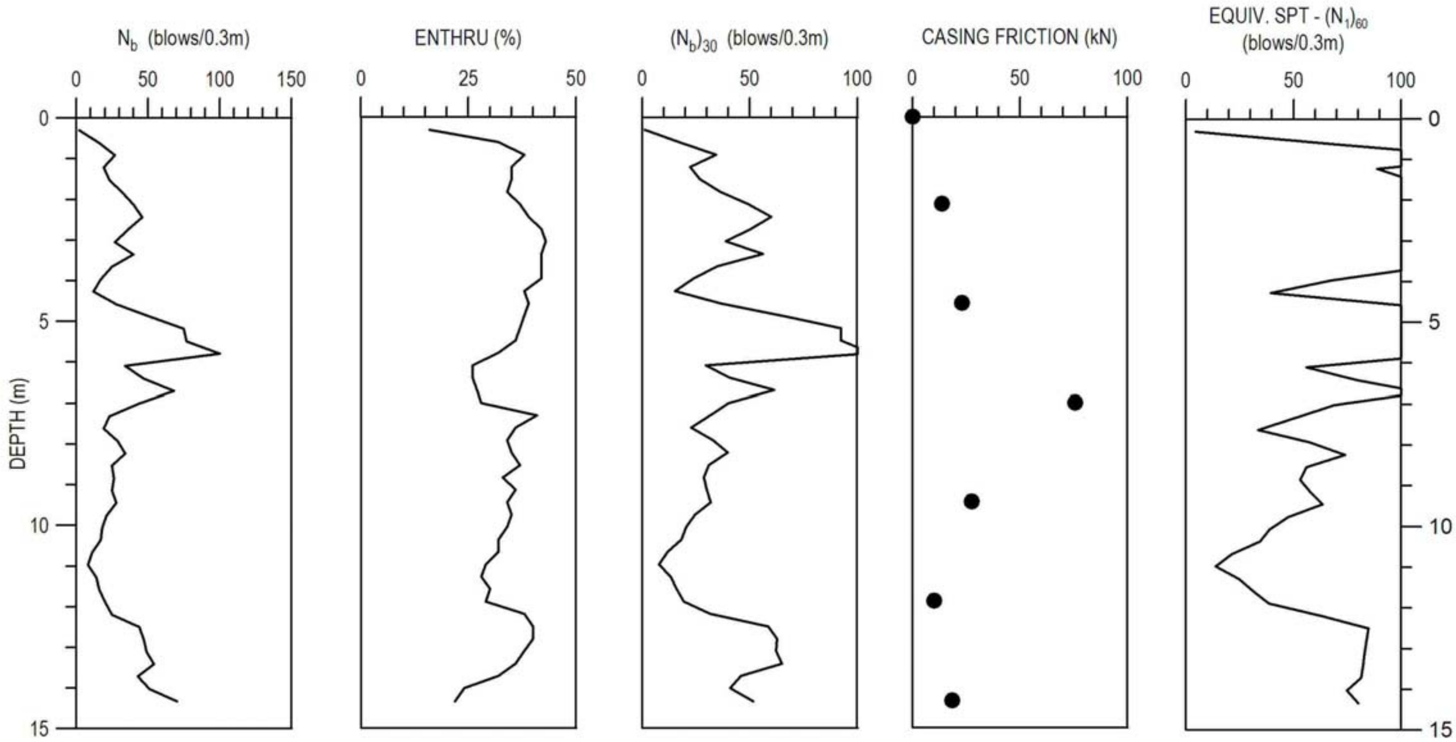
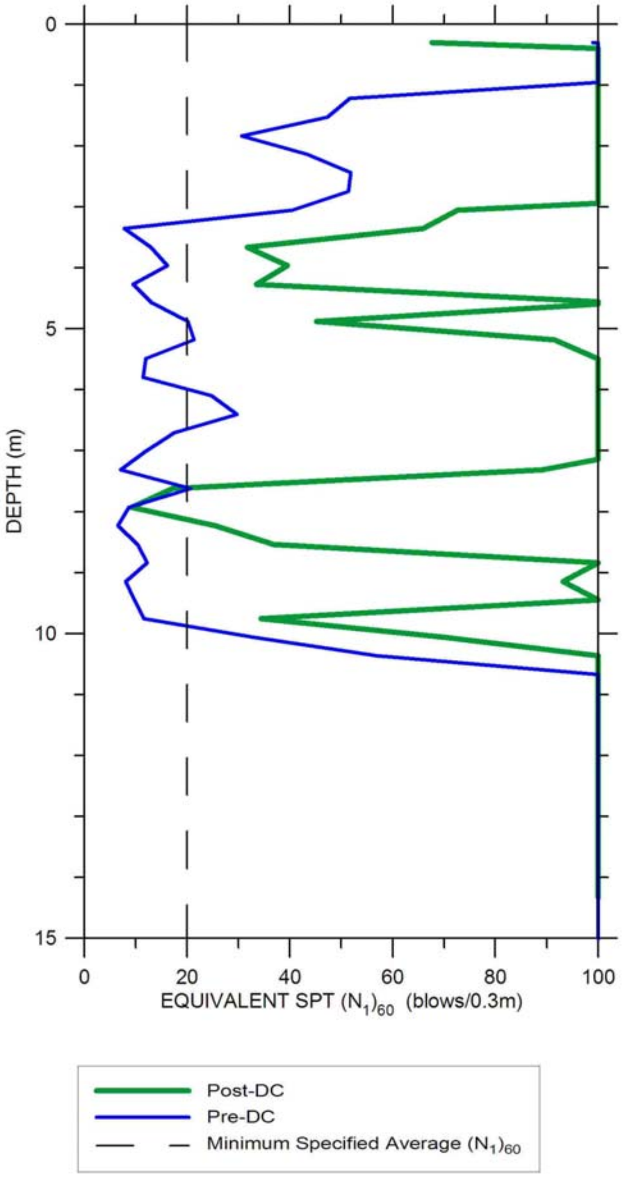


Fig. 10 – Results of post-DC BPT at Site B



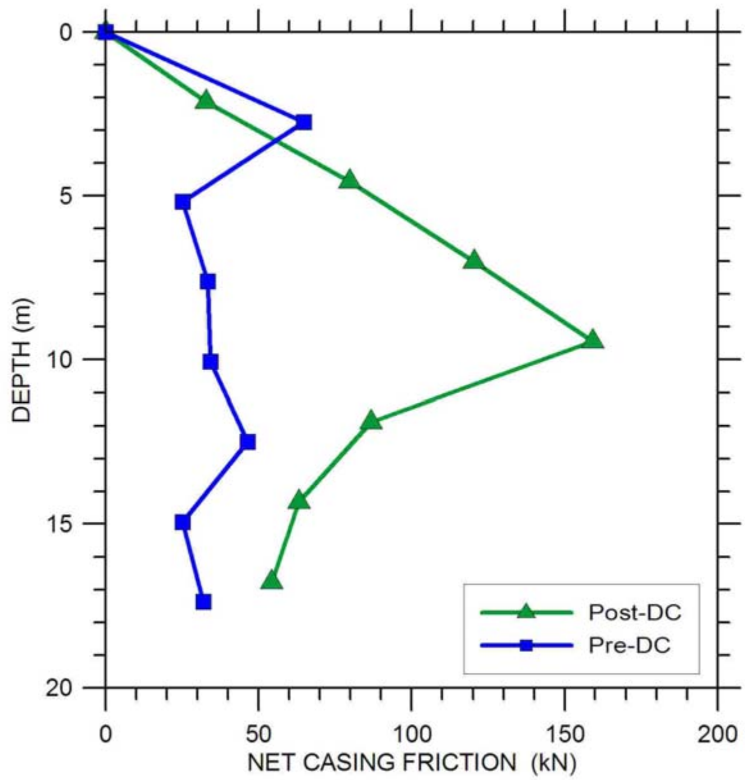


Fig. 11 – Pre-DC and Post-DC $(N_1)_{60}$ Profiles in Zone I at Site B

Fig. 12 – Typical BPT Casing Friction Profiles in Zone I at Site B