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Composite Ground Modification System: Vibroreplacement and Dynamic Compaction, Salt Lake County Detention Center, Utah

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ABSTRACT

The site for a new Adult Detention Center currently under construction in Salt Lake City, Utah, is underlain by loose sands and soft clayey lake deposits. Due to bearing capacity, static settlement, and liquefaction concerns, a hybrid ground improvement program consisting of both dynamic compaction and vibroreplacement was implemented. Stone columns were concentrated under spread and wall footings; dynamic compaction was performed over the whole site. A comprehensive quality assurance / quality control program was executed, with a significant number of cone penetration tests, standard penetration tests, 10 plate load tests, and deceleration readings taken with an accelerometer mounted on the dynamic compaction weight. This large body of data enabled the authors to assess the effectiveness of the ground improvement program, as well as analyze the results of the experimental deceleration readings.

KEYWORDS

vibroreplacement, stone columns, bottom-feed, dynamic compaction, ground improvement, ground modification, load test, accelerometer, cone penetration test, standard penetration test, Adult Detention Center, unloading point method

INTRODUCTION AND BACKGROUND

A new Adult Detention Center is currently under construction for Salt Lake County approximately 8 km southwest of downtown Salt Lake City, Utah. The complex will consist of 4 general population pods (9000 m^2 each), a jail support building (7000 m^2), a food service building (4400 m^2), and a central plant building (3000 m^2). The primary structures will be two stories in height and will consist of precast and cast-in-place concrete walls with slab-on-grade floors.

The site is located within the central portion of Salt Lake Valley, flanked by two uplifted range blocks, the Wasatch Range and Oquirrh Mountains. The soils consist of deep water lake deposits of clay, silt and fine sand. More specifically, the natural soils consist of ½ to 1 meter of organic clay and silt (OL) overlying a layer of loose to medium dense sand (SW-SM), silty sand (SM) or sandy silt (ML), which in turn overly a series of lean clay (CL) and organic clay (OL) layers. Depth to groundwater varies seasonally from less than 1 meter in the spring to 3 meters in the fall. Prior to construction, the sand layers exhibited a wide range of blows between 4 and 43, indicating potential for liquefaction in some areas of the site.

Both the Architect/Engineer and Geotechnical Engineer concluded that the difficult soils presented a foundation challenge for the relatively rigid structures. Three concerns were identified: net allowable bearing capacity, gross and differential static settlements, and loss of support / settlement during the design seismic event. The following six foundation alternatives were considered:

1. concrete mat,
2. pile foundation,
3. overexcavation and replacement,
4. raised structural fill,
5. vibroreplacement (VR),
6. dynamic compaction (DDC).

All six alternatives were evaluated for technical merit (reliability of system to meet foundation performance requirements), cost, time to implement and construct, and dewatering requirements. Other indirect impacts were considered, such as: suitability for future construction/expansion, effect on public, effect on adjacent structures, impact on future utilities, and ability of system to absorb design changes concurrent with and after construction.

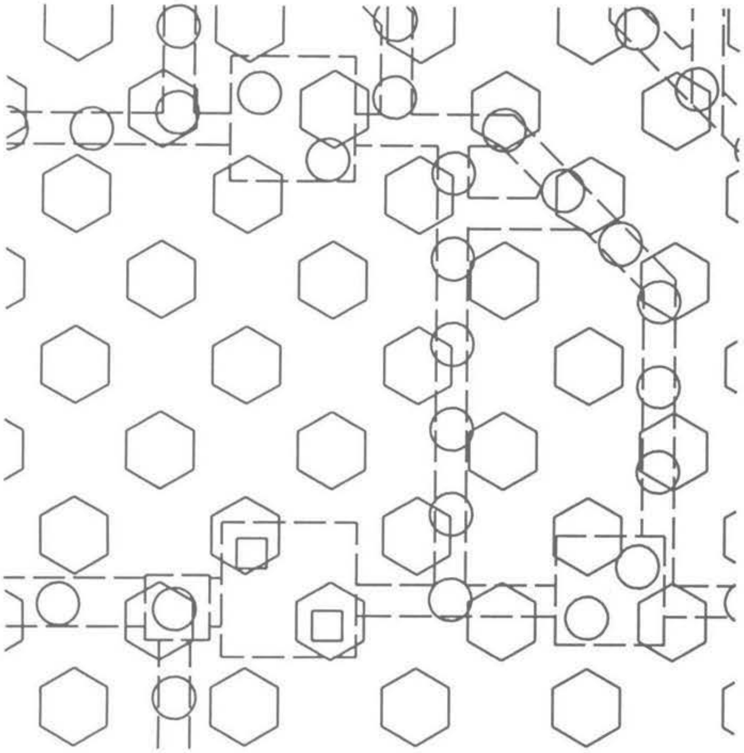

Fig. 1 Selected section of the building showing layout of dynamic compaction and stone column points.

GROUND IMPROVEMENT SOLUTION

A performance specification was drafted that required a post ground improvement bearing capacity of 144 kPa and maximum total settlement of 25 mm. To guarantee seismic performance, the specifications also required mitigation of the liquefaction potential of the granular soils, to be corroborated by a minimum cone penetration resistance of 11,500 kPa for soils with less than 1.5% friction ratio.

The original ground improvement program called for stone columns installed on a grid pattern using the dry, bottom-feed vibroreplacement method under the buildings' footprint plus a 6 meter perimeter. After several bidding iterations and in an effort to reduce the project's costs and time of construction, the Specialty Subcontractor proposed a solution that combined two methods of ground improvement: dynamic compaction and dry, bottom-feed vibroreplacement stone columns. Under this design, the stone columns were concentrated under the spread and strip footings to a depth equal to 2B and 4B respectively to comply with the bearing capacity and settlement parameters, as well as liquefaction mitigation under the footings. The remainder of the site, namely the slab areas and perimeter of the structures, were treated only for liquefaction using dynamic compaction. Note that because the granular soils occur only in the top 5 meters, dynamic compaction was ideally suited for this site. The clayey nature of the underlying layer, however, required the reinforcing effect of the stone columns to properly design against settlement.

CONSTRUCTION PROCEDURE

Prior to ground improvement the superficial organic clay layer was removed. Then vibroreplacement was performed using the Keller "S" vibrator, equipped with side pipe and pressure chamber to feed gravel to the bottom of the hole. Under this bottom-feed process no jetting water is used; the hole is created solely by the vibrations of the probe and the weight of the heavy-walled extension tubes. Upon reaching design depth, the vibrator is retrieved in 1.5 m lifts to allow placement of backfill, and the stone is repenetrated. The process is repeated until ground surface is reached, forming a well compacted gravel column, or "stone column", and densifying the surrounding soils due to the combined effects of vibrations and lateral displacement. The "S" vibrator subjects the ground to 26 tons of rotating force at a frequency of 30 Hz.

Two complete vibro setups were used to carry out this project. A third crane was rigged with a 12 ton weight to perform dynamic compaction. The weight was dropped 3 to 5 times from heights of 20 m on a predetermined diamond shaped grid of 2.4 m. Applied energy levels varied from 156 to 312 ton - m / m^2 depending on soil conditions and whether ice had formed on the ground.

Dynamic compaction was performed on a grid pattern after installation of stone columns over the entire treatment zone, including areas where stone columns had been installed. After dynamic compaction, a tamping pass was performed with a flat 10 ton weight measuring 2.1 m by 2.1 m by 0.25 m dropped from a height of 10 m. In this fashion, any near surface soils that may have been loosened by dynamic compaction or vibroreplacement were densified.

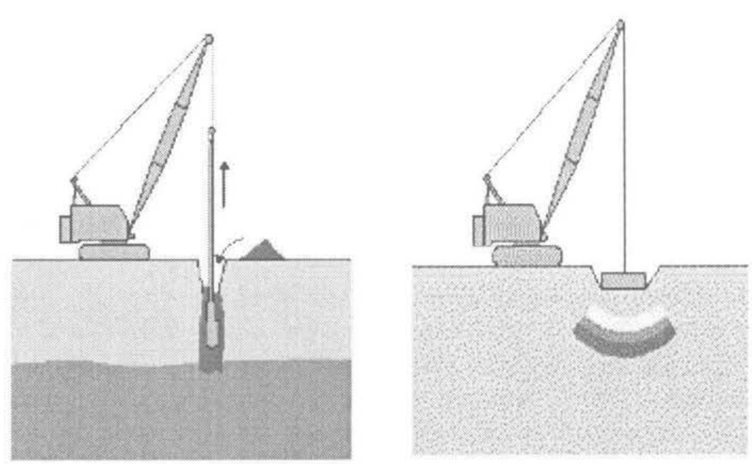

Fig. 2 Sketch showing typical vibroreplacement and dynamic compaction operations.

QUALITY CONTROL / QUALITY ASSURANCE

Quality control of ground improvement was performed by several different methods. CPT tests were conducted every 460 m^2 and SPT tests every 2300 m^2. In addition, 10 plate load tests were performed both in areas treated by stone columns and by dynamic compaction. Vibration monitoring was performed to ensure protection of nearby buildings and utilities. Lastly, an accelerometer was mounted on the dynamic compaction weight at select locations to observe the effectiveness of dynamic compaction on ground stiffness and to compare with the load test results.

Post treatment CPT results have satisfied or exceeded the specified minimum resistance, though certain areas required secondary treatment with dynamic compaction. Some locations were tested more than once, exhibiting increases in tip resistance with time.

In areas where the required CPT resistance was not initially achieved, additional "remedial" drops were performed in between the original work. Remedial work occurred mostly toward the end of the project as winter progressed, and the combination of snow, frozen ground, and rise in the water table made compaction of the siltier soils difficult. Remedial work consisted of 3 additional drops from a height of 20 m, bringing the total applied dynamic compaction enegy levels in those areas to 312 to 416 ton - m / m^2.

While post treatment minimum resistance criteria provides a level of confidence for anticipated foundation performance, it is often desirable to compare pre and post treatment penetration resistance as an indication of the level of improvement achieved.

Though predominant trends of stratigraphy were identified during the geotechnical investigation, soft compressible deposits in the Salt Lake Valley have been found to vary significantly within relatively short distances. This can obscure attempts to compare individual pre and post treatment resistance logs taken at nearby locations. Therefore, averaging records when sufficient data is available provide a more consistent representative comparison.

CPT tip resistance data from 7 pre improvement locations and 43 post improvement locations have been averaged and are compared in Figure 3. Note that some improvement occurred in the saturated clayey silt materials. We believe that some limited drainage was afforded during dynamic compaction by the overlying sand layers and interbedded sand lenses within the clayey silt. The generalized soil profile shown in Figure 3 is taken from SPT logs, which provide verification of fines content.

SPT testing was performed primarily to verify the stratigraphy and fines content of the subsurface materials. Fines content generally increases near the interface between the sand and silt on this site. While less extensive than CPT test data, comparison of pre and post improvement SPT resistance may also be useful

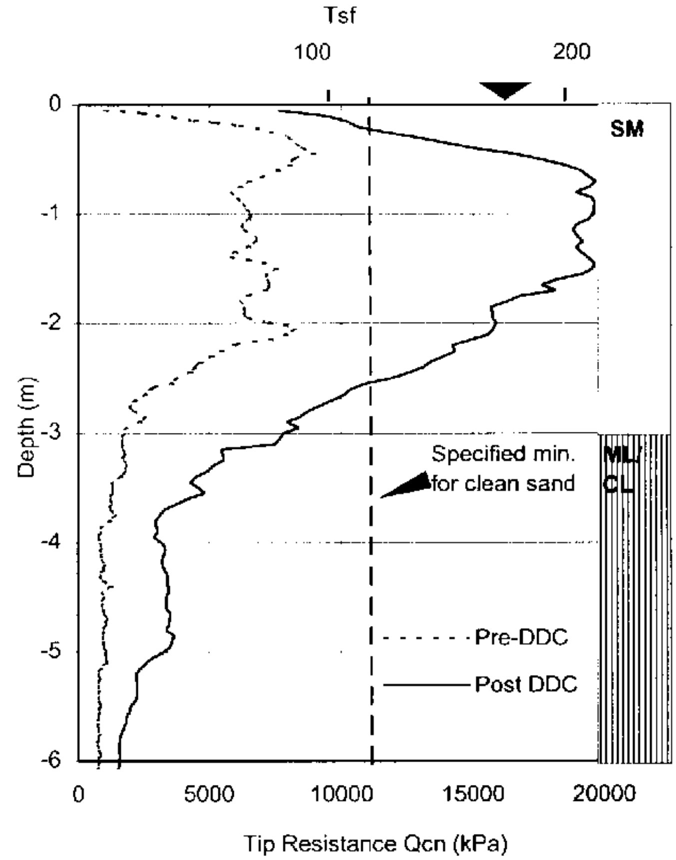

Fig. 3 Comparison of average CPT normalized tip resistance, Pods A and C, 7 pre-DDC locations, 43 post-DDC locations.

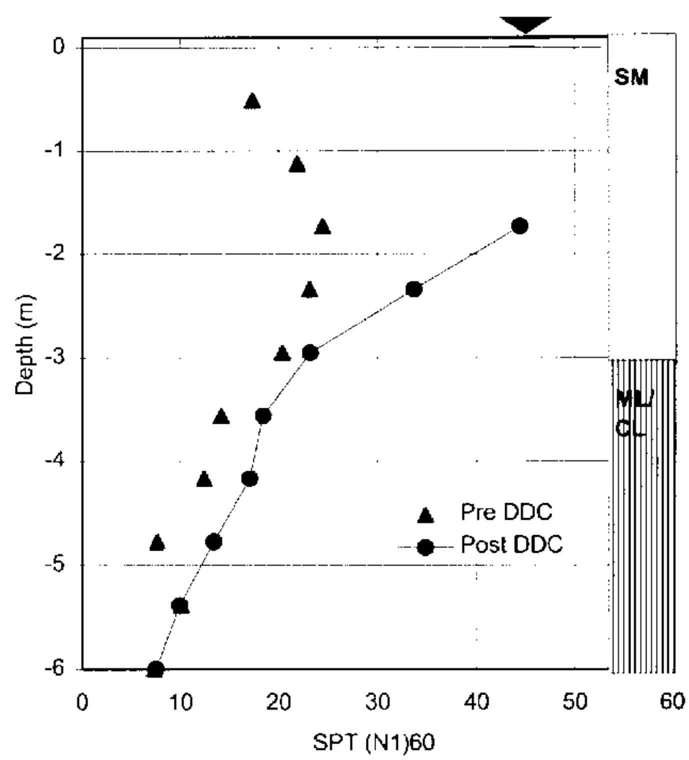

Fig. 4 Comparison of average SPT resistance, all pods and central support, 17 pre-DDC borings, 24 post-DDC borings.

by averaging data for a given area. Results from borings in all four pods and the central support building are given in Figure 4. Again, some limited improvement in the saturated silt is evidenced.

Plate load testing also satisfied specified stone column and dynamic compaction performance criteria: 25 mm deflection under the design bearing capacity and 50 mm under a load of twice the allowable bearing capacity. Design bearing capacity in stone column areas (footings) was 144 kPa; and in DDC areas (slabs) was 34 kPa. Load tests were taken to twice the design bearing pressure using a 60 and 36 inch plate for stone column locations and dynamic compaction locations, respectively. Loading period was 24 hours, in general accordance with ASTM D1194 procedures.

Results from load tests in Pod C are given in Figure 5. For the 10 plate load tests at twice the design load, stone column tests had a maximum deflection of 45 mm, with an average of 20 mm, and DDC tests had a maximum deflection of 21 mm , with an average of 8 mm.

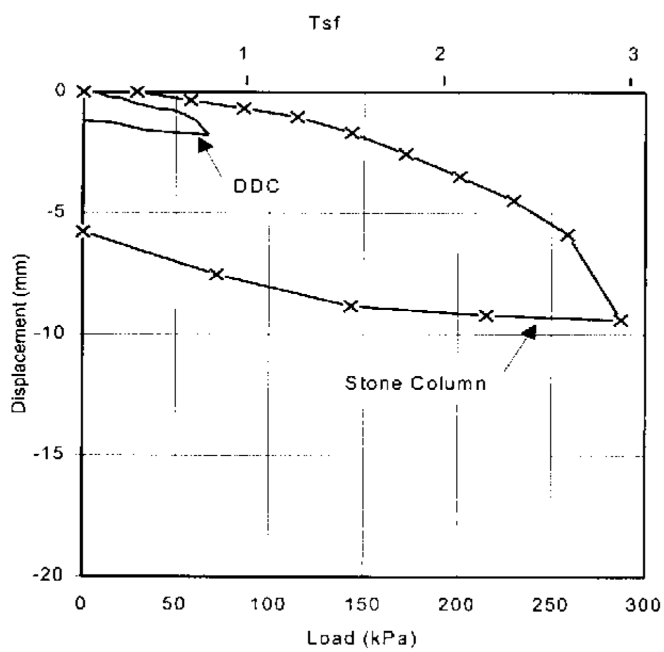

Fig. 5 Static plate load tests results, Pod C; DDC Location C114, Qult=67 kPa; Stone Column Location C600, Qult=287 kPa.

DECELERATION MEASUREMENTS

In an effort to advance the state-of-the-practice in dynamic compaction, deceleration measurements were made at a number of drop locations near the site of the plate load test for Pod C. The deceleration measurements were then analyzed to predict the measured load-displacement response. In addition, the deceleration measurements were used to determine when pore pressure build-up was reducing the effectiveness of the compaction.

Deceleration measurements were made using an impact resistant "on-board" data acquisition system which eliminated the need for any connecting wires. The system consisted of two accelerometers, a battery pack, a data acquisition unit, and a radio receiver. These components were housed within a steel box about 30 cm square and 10 cm high which was attached to the center of the tamping weight. The radio receiver was used to turn the data acquisition system on immediately prior to impact by remote control. This procedure made it possible to record about 15 drops at a rate of 2400 samples per second before downloading the data to a laptop computer. The system was ruggedized to withstand approximately 500g.

The acceleration time histories for four sequential drops at a location that corresponds with the plate load test in Pod C are presented in Figure 6. As the tamping progresses, the width of the acceleration pulse (i.e, the period) tends to increase partly due to the development of the crater and the acceleration peak tends to decrease due to the development of excess pore water pressure. The drop in peak acceleration is particularly evident on the third drop.

The penetration of the tamper was measured after each drop. Using this value as an integration constant, time histories of velocity and deceleration could be accurately computed. These time histories are shown for one drop in Figure 7. The computed velocities were then used to calculate drop efficiency, which was reasonably constant at about 80%. Peak acceleration, acceleration period, drop efficiency, and tamper penetration for each drop and an ironing pass are summarized in Table 1.

Several researchers (Lukas, 1986; Poran and Rodriguez, 1992) have made efforts to evaluate the improvement achieved through DDC using deceleration measurements, however success has been limited. Francis (1996) recently developed a procedure for computing the equivalent static load-displacement curve from the deceleration time history using a modification of the unloading point method (Brown, 1994). Using this approach, the damping force is subtracted from the measured inertia force to obtain the soil (spring) force as a function of time.

The equivalent static soil pressure versus displacement computed using this method is shown in Fig. 8 for four drops at one location. The pressure versus displacement curve measured during the static plate load test is also shown in Fig. 8. The maximum static pressure increases with the number of drops until drop four. At this point, the excess pore pressures have built up and the tamper appears to punch through as evidenced by the increase in measured displacement and the drop in maximum soil pressure.

Previous experience has shown that once punching occurs, localized shearing from low effective stresses will inhibit subsequent drops from achieving efficient densification. Should additional densification be required, it is more productive to allow a waiting period to allow dissipation of excess pore pressures prior to continued tamping.

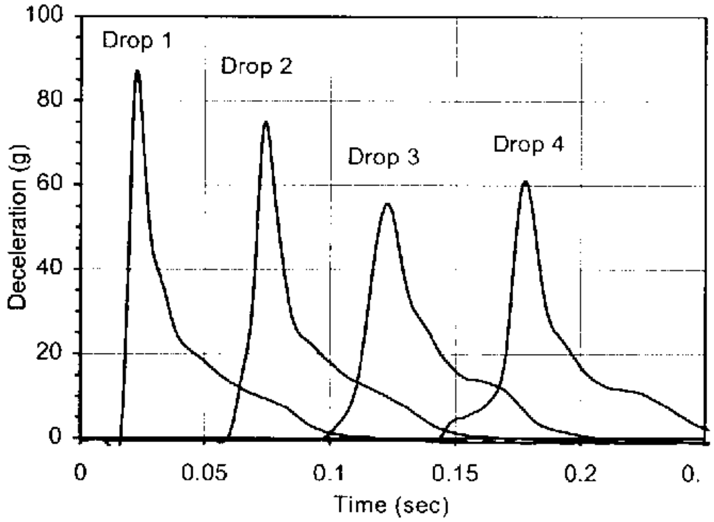

Fig. 6 DDC impact response deceleration, Pod C, Location C114, Drops 1-4.

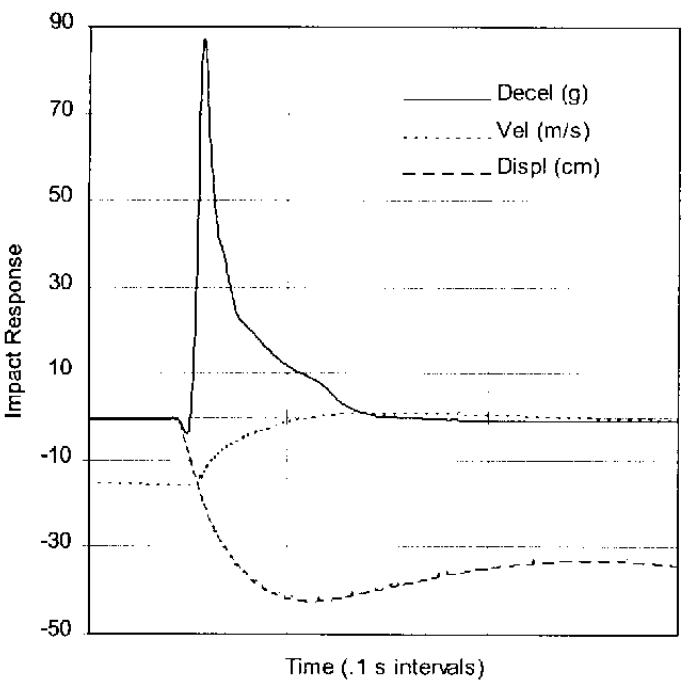

Fig. 7 Comparison of DDC impact response deceleration, velocity, and displacement, Pod C, Location C114, Drop 1.

Figure 8 also shows that the rebound portion of the computed pressure-displacement curves are similar to that measured by the plate load test, which was performed three weeks after dynamic compaction. The rebound moduli for each drop during the loading range from 0 to 100 kPa are tabulated in Table 1 and they are in reasonable agreement with the measured plate load rebound modulus of 114 kPa / mm. It should be noted that the computed moduli actually decrease with the number of drops apparently due to the build-up of excess pore water pressure.

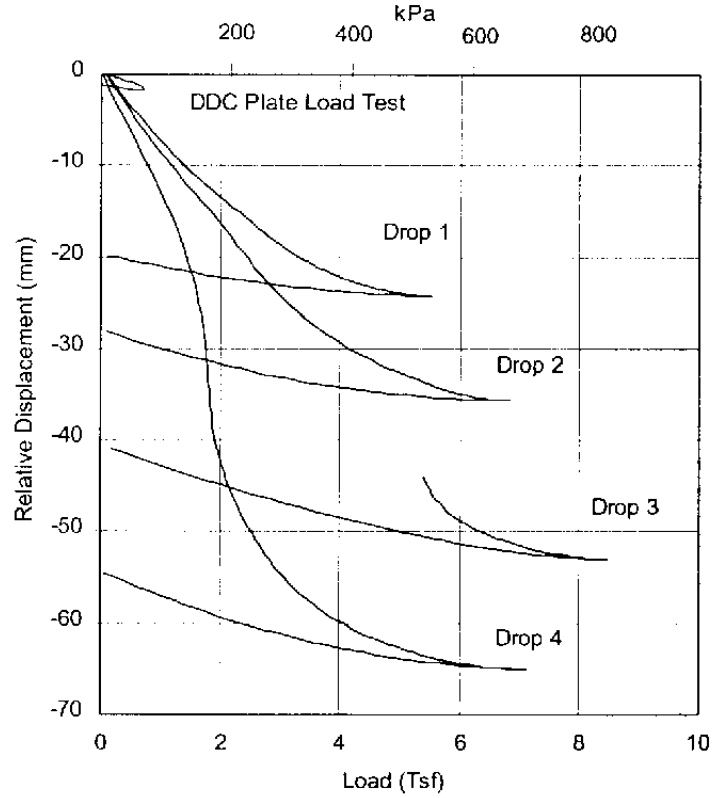

Fig. 8 Static DDC plate load test compared with equivalent static load rebound curves, Pod C, Location C114, Drops 1-4.

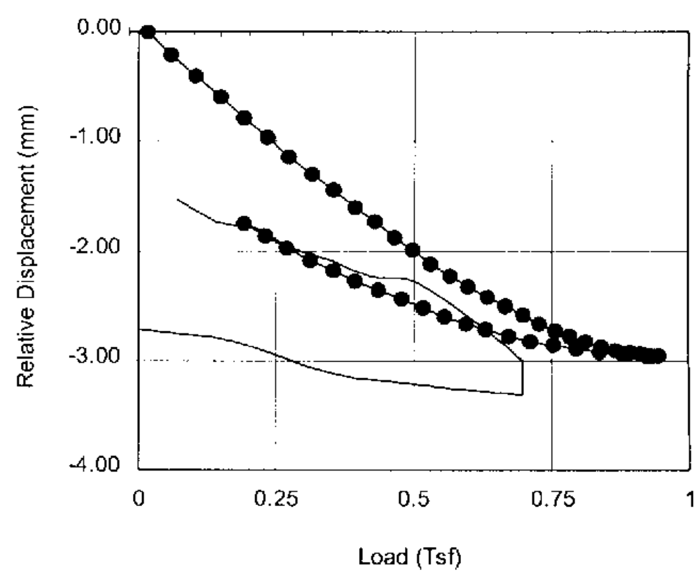

Fig. 9 Static DDC plate load test compared with equivalent static load rebound curve, Pod C, Location C114, Iron Drop.

A relatively low modulus was measured during the iron drop, which may be attributable to the infolding of craters with loose soil between the heavy tamping and ironing phase. Data from the ironing pass also confirms substantially lower applied static loading than during heavy tamping, even though peak deceleration is comparably high.

It is interesting to consider that each drop during dynamic compaction constitutes a successive loading cycle, and that the plate load test constitutes an additional successive cycle, following the iron drop. Data from the plate load test is presented as a cycle following the iron drop in figure 9.

Since the plate load test was applied over a lower stress range than dynamic compaction, the soil which is now over consolidated would ideally exhibit a loading modulus which equals the rebound modulus, and no permanent set would occur. However, as demonstrated in figure 9, effects of nonlinearity, seating of the load, and hysteresis result in a loading modulus which may range from 114 kPa / mm to as low as 38 kPa / mm, depending upon the method and load range used to compute the modulus.

From the range of plate load moduli, it appears that the modulus computed from the iron pass provides a reasonable estimate of a lower bound, and agrees with the authors current opinion that the unloading point method generally provides a conservative estimate of static bearing capacity and soil matrix stiffness.

The results from this study suggest that deceleration measurement may eventually lead to a procedure for evaluating improvement in "real-time" during compaction and for predicting when excess pore pressures are hindering soil improvement, by determining maximum static loading, settlement, and rebound stiffness. Other dynamic criteria such as peak deceleration, period, and damping assist with the evaluation. Additional comparative studies need to be conducted to refine and verify the procedure.

DDC Drop #	Peak Decel. (g's)	Period (sec)	Vel. Dmpng Coeff. (C)	Calc. Drop Efficie-ney (%)	Survey Vertical Displa-cement (cm)	Max. Equiv. Static Load (kPa)	Rebnnd Mdulus (kPa)/ mm
1	88	.9	26	80	46	530	68
2	75	.1	25	79	22	655	45
3	55	.11	18	76	17	810	42
4	61	.12	22	76	24	682	34
Iron	70	.1	23	82	18	91	39-76
Plate Load	NA	NA	NA	NA	.176	67	38-114

Table 1 DDC Impact Response Parametric Comparison

CONCLUSIONS

The combination of dynamic compaction and vibroreplacement proved to be the most advantageous solution for this site from both technical and economic perspectives. The combination of methods resulted in a 40% cost reduction had the whole site been treated with vibroreplacement, and schedule was shortened by 5 weeks. From the Owner's perspective, a relatively nominal cost afforded significant assurance that this critical structure will function as intended over its lifetime.

Because the sands that needed compaction were relatively shallow, the levels of densification achieved with dynamic compaction were excellent. The average of 7 pre improvement CPT's and 43 post improvement CPT's yielded a factor of improvement of 2.9 as tip resistance increased from 7000 kPa to almost 20,000 kPa. The silts and clays between 3 and 5 meters experienced a surprisingly high level of improvement, as tip resistance increased from 1100 kPa to 3200 kPa, yielding an average improvement factor of 2.9, equal to that of the sands. Although the dynamic compaction energy levels were designed for a depth of 5 m, improvement was still evident at a depth between 5 and 6 m, as tip resistance more than doubled from 1100 kPa to 2800 kPa. A similar, though less marked, trend was evident in the SPT readings.

Plate load tests were performed to verify allowable bearing capacity. A comparison between plate load test data and deceleration measurements was made to evaluate improvement and effects of excess pore pressures on soil improvement. Equivalent maximum static loading, settlement, and rebound stiffness are the primary results from the analysis of deceleration records. Once developed, the procedure would allow one to determine the optimum number of drops per location prior to inducing excess pore pressures, and will help establish predicted deflections for a given applied load.

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REFERENCES

Brown, D.A., [1994]. "*Statnamic Test Analysis*", ASTM Journal of Geotechnical Testing, ASTM, Philadelphia, pp. 403-414.

Francis, M.J., [1996]. "Effects of Moisture Content on Dynamic Compaction with an Evaluation of Quality Assurance Methods", Master's Report, Brigham Young University, Provo, Utah.

Lukas, R.G., [1986]. "*Dynamic Compaction for Highway Construction, Vol I: Design and Construction Guidelines*", Report No. FHWA/RD-86-33, Federal Highway Administration, U.S. DOT, Washington, D.C., pp 204-216.

Poran, C.J, and Rodriguez, J.A., [1992]. "A New Technique for Quality Control of Dynamic Compaction", Grouting, Soil Improvement, and Geosynthetics, ASCE Geotechnical Special Publication No. 30, New York, NY. pp. 915-26.