Soil improvement by combining methods

L'amélioration des sols par diverses méthodes combinées

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SYNOPSIS: On several recent projects the combination of two or more ground treatment methods has provided an economical and efficient means for achieving the needed soil improvements. Four such projects are described herein. They include the use of vibrocompaction over deep blasting, dynamic deep compaction over compaction grouting, stone columns and deep dynamic compaction, and a four-method project in which vibro-compaction, vibro-replacement, compaction grouting, and deep dynamic compaction were all used effectively.

INTRODUCTION

In the eight years since the first comprehensive review and discussion of soil stabilization and ground improvement methods as a main topic at an International Conference of this Society, both interest in the subject and new methods and applications have continued to develop rapidly. Each of the many methods for soil improvement has unique advantages, limitations, and applications relative to different soil types, ground conditions, and project requirements, as summarized by Mitchell (1981,1988) and others.

For a number of important recent projects the required soil improvement could not be accomplished, for practical or economic reasons, by using only a single method, but by combining two or more methods effective, efficient, and economical results were achieved. The purpose of this paper is to illustrate, by means of four case histories, how combined methods of ground improvement can be used to advantage.

VIBRO-COMPACTION OVER DEEP BLASTING: THE JEBBA HYDROELECTRIC DEVELOPMENT

The main dam at the Jebba Hydroelectric Development in Nigeria, shown in cross section in Fig. 1, is founded on clean alluvial sand having a maximum depth to bedrock of more than 70 m. The maximum height of the dam is 42 m. The sand required in-place densification to minimize settlements during construction and reservoir filling and to preclude liquefaction of the looser zones in the event of earthquake. Cracking in either the clay core or in the compacted clay seepage control blanket, which extends 450 m upstream from the upstream toe of the embankment, was to be prevented.

The clean, fine to coarse river sand has a mean grain size of about 1.0 mm and an average uniformity coefficient of 2.94. It was required that minimum equivalent relative densities be attained of (1) 70 percent in the top 20 m layer, (2) 60 percent in the next 10 m layer,

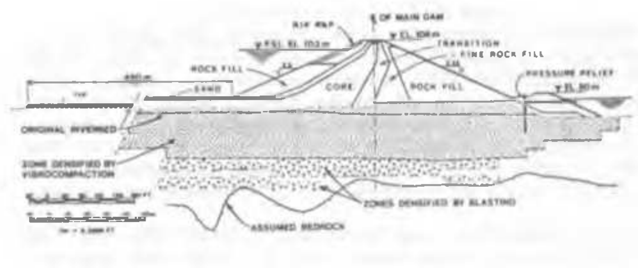

Fig. 1 Cross Section of the Jebba Main Dam, Nigeria, Showing Zones Densified by Blasting and Vibro-Compaction

and (3) 50 percent at depths greater than 30 m below original river bottom. The concept of "equivalent relative density" was developed for this project, and is defined as follows: a given sand has an equivalent relative density P% if it gives a static cone penetration resistance equal to that for the same sand freshly deposited, saturated, and normally consolidated at the actual relative density P%. The intent is to recognize that the cone resistance reflects the influences of factors other than relative density; e.g., aging, overconsolidation, and that the cone resistance is a more reliable indicator of strength and liquefaction resistance properties than is the relative density by itself.

The required depth of treatment at this site was unprecedented. Although it seemed probable from the outset, on the basis of prior experience, that densification to depths of 25 to 30 m could be possible using vibro-compaction, no techniques were available for greater depths. Accordingly, deep blasting was chosen as the one possible means for accomplishing the needed treatment.

Following a test program five zones beneath the left half of the river were designated for blasting, as shown in Fig. 2. Charges of up to 30 kg were used, with a loading factor of

Fig. 2 Plan of Jebba Main Dam Area Showing Zones to be Densified by Deep Blasting

25-35 gm explosive per m^3. Three coverages were used for zones 1-4, and 4 coverages were required in zone 5. The charges for each coverage were placed in the zones shown in Fig. 1 at 10 m spacings in a square pattern. Ground surface settlements of up to 1.10 m were measured as a result of the blasting program. The improvement in relative density averaged over the full depth varied between 5 and 16 percentage points, with an average of 10 percentage points.

Vibrocompaction was done using both 100 and 130 horsepower probes operating at 1800 rpm and at 2.75 m probe spacings. Depths of treatment using vibrocompaction ranged from 10 m to 35 m. The river sand was used as backfill. The average production rate varied from 300 m to 600 m per probe per 10 hr shift, but sometimes reached 1000 m. The specified improvement by vibrocompaction was readily attained in most areas.

The overall foundation treatment for the Jebba project is described in more detail by Solymar et al. (1984), and the deep blasting program is described by Solymar (1984).

Experience at this project showed clearly that freshly deposited or densified clean sand may develop substantial stiffening and strength increase with time for periods extending over several months. Such deposits may also undergo significant loss in strength as a result of disturbance, thus behaving in some respects in a manner similar to sensitive clay. This behavior is described and analyzed in more detail by Mitchell and Solymar (1984) and Mitchell (1986,1988). This time-dependency has now been observed to influence the penetration resistance of different sands, to varying degrees, for all types of deep densification; i.e., blasting, vibrocompaction, dynamic compaction, compaction piles. An example from one of the blasted zones at the Jebba project is shown in Fig. 3.

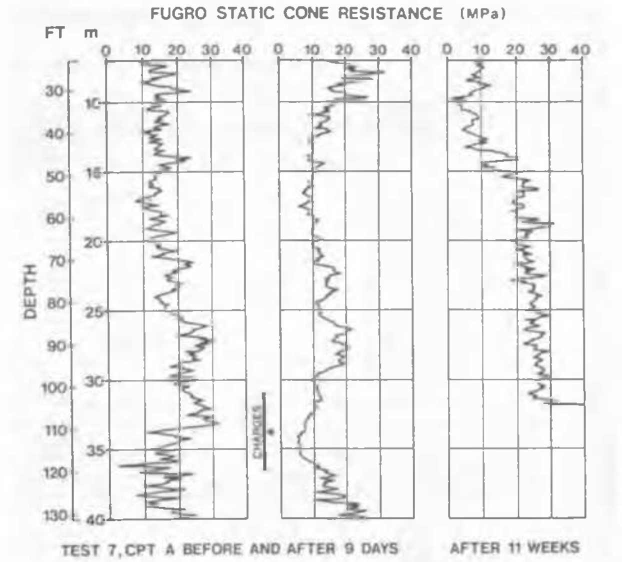

Fig. 3 Penetration Resistance of Thick Sand Layer at Two Times After Densification by Blasting at the Jebba Dam Site

COMPACTION GROUTING AND DEEP DYNAMIC COMPACTION: ST. JOHNS RIVER POWER PARK PROJECT

The 8.5 hectare site at the St. Johns River Power Park near Jacksonville, FL is a low lying naturally filled-in former marine estuary. The generalized soil profile consists of: (1) very loose to dense fine sand with "N" values between 5 and 25 to 10 meters depth; (2) material similar to Layer 1, with "N" values between 15 and 70 to a weak contact zone at 15 meters depth, which consisted of a 3 meter thickness of voids or loose material filling former voids; and (3) cemented and partially cemented sands and shells.

A pile foundation would have required piles driven into Layer 3. Three methods were proposed for ground improvement: (1) deep dynamic compaction of the upper sands and stone columns in the weak contact zone; (2) vibro-compaction in the upper sands with stone columns in the weak contact zone; and (3) deep dynamic compaction in the upper sands with compaction grouting in the weak contact zone. Alternative (3) was selected as more economical than either of the other two alternatives or pile foundations.

The deep dynamic compaction was done using a 32,700 kg weight dropped from 30 meters, 2 to 7 times per print location, with the primary pass on a 10 meter grid spacing, and the secondary pass located in the center of the primary grid. Compaction grouting of the weak contact zone was accomplished by jetting the grout pipes to the top of Layer 3 and then injecting compaction grout at pressures up to 7 MPa. The grout consisted of an automatically proportioned mixture of site sand, fly ash, cement and water, with a slump of approximately 75 mm.

Time improvement factor effects for the q_c values of the cone penetration tests used to verify that 85 percent relative density had been attained are well documented in Schmertmann, et al. (1986). Other aspects of the project are discussed by Kessler and Kuretski (1985) and Welsh (1987).

STONE COLUMNS AND DEEP DYNAMIC COMPACTION: STEEL CREEK DAM

Steel Creek Dam is located in an active earthquake area in South Carolina, with the dam design based on an earthquake acceleration of 0.1 g caused by a magnitude 6.6 event. Subsurface soil profile conditions are shown in Fig. 4. The embankment dam is 670 meters long, with a maximum height of 27.4 meters. The base of the dam sits on a loose clayey sand extending to a depth of 21 meters with "N" values less than 10 blows per 0.3 m.

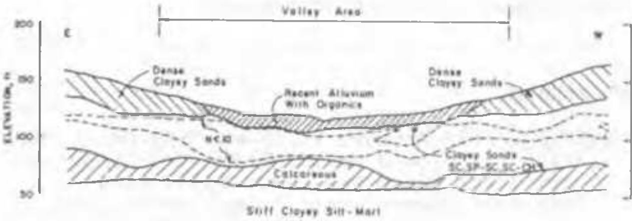

Fig. 4 Simplified Longitudinal Subsurface Profile at the Steel Creek Dam (From Keller et al., 1987)

Methods evaluated for improvement of the loose clayey sand included deep dynamic compaction, stone columns, compaction grouting and vibratory pile driving (Castro, et al., 1987). Vibratory pile driving caused some densification to occur but tended to mix soil layers and make densification evaluation difficult. Compaction grouting was generally ineffective in densifying the loose clayey sand.

Deep dynamic compaction was selected to densify the valley section of the dam, with dry bottomfeed stone columns used on the abutment sections and the deeper portion of the valley, where the required depth of treatment was greater than could be attained using dynamic compaction. The deep dynamic compaction of the 18,000 square meter valley section to a depth of 9 meters using a 27,200 kg freefall weight dropped from 30 meters is described by Dobson (1986).

Dry bottomfeed stone columns were also used on 2,230 square meters near the downstream toe of the dam where the loose soil was too deep to be densified by deep dynamic compaction. Four specially fabricated dry bottomfeed vibratory units were constructed to install 42,670 linear meters of stone columns to a maximum depth of 21.3 meters. To penetrate the stiff upper layer of dense clayey sand, a continuous flight auger for drilling was attached to the side of the leaders. The bottomfeed vibrator was placed in the predrilled hole and the stone fed to the nose of the vibrator through a 15 cm diameter feed pipe pressurized with compressed air to ensure flow of the stone backfill to the bottom of the vibrator.

In general the dynamic compaction and stone columns were effective in densifying clayey sands with less than 10 percent clayey fines, but not effective in densifying sands with larger percentages of clayey fines.

A FOUR METHOD PROJECT: THE TRIDENT SUBMARINE FACILITY

There was major concern for settlement and liquefaction at the expanded submarine base at Kings Bay, Georgia. The soils in the upper 15 meters are recent sedimentary deposits comprised of normally consolidated sands, silts and clays, with variations in soil types encountered over short distances. Common to the coastal area and present at the site is a near surface layer of dense, cemented, fine sand, referred to as 'hardpan', which normally has low blow count sands beneath it (Table 1) A seismic risk analysis indicated that four significant earthquakes had affected the site area since the 1800's. The facility was designed for a peak ground acceleration of 0.1 g.

TABLE 1

Generalized Subsurface Profile (Pre-Treatment) at the Trident Submarine Base, Kings Bay, Georgia

Depth (m)	Soil Description	Typical Range of Test Values: SPT "N" Value (blows/0.3 m)	CPT Tip Resistance (MPa)
2.5	Fine Sand (SP, SP-SM)	2-50+	2-30
2.5-4.5	Cemented Organic Stained Fine Sands (SP, SP-SM)	31-100+	10-50+
4.5-5.5	Silty Fine Sand (SM)	2-9	0.5-5
5.5-15	Fine Sand with Silty Sand Sand & Clayey Sand Layers (SP-SM, SM and SC)	1-40	0.5-25

The project comprised 6 major building sites plus 39 storage areas. It was specified that the cohesionless soils be densified to 70% relative density and, in the case of cohesive soils, the improvement criterion was to limit the total settlement to a maximum of 12.7 mm. The method of soil improvement was left to the contractor's discretion.

To achieve the required density and/or settlement criteria, the site improvement contractor considered vibro-compaction, stone columns, compaction grouting and deep dynamic compaction (Hussin and Ali, 1987). The initial design, coupled with the contractor's supplemental electric cone penetrometer and standard penetration tests, and the scheduling of equipment, resulted in these techniques being used individually or in combination to reach the required specifications.

The 17,000 square meter Missile Motor Magazine area required improvement to a depth of 9.1 meters. Deep dynamic compaction was not permitted in this area because of adjacent utilities. Therefore, vibro-compaction was initially tested, but it was found that even using a large 165 HP vibrator, the 1 to 1.5 meter thick silty sand immediately below the hardpan could not be densified efficiently. Therefore, stone columns were used, the points being placed 2.7 m center to center in a square grid. Where pretesting determined very thin strata of loose soil, compaction grouting was utilized, since it was determined that injecting the very stiff mix under high pressure could adequately densify thin, loose zones even when grout pipes were placed in a 2.7 meter primary/secondary grid pattern.

At the 6,000 square meter Radiographic Inspection Building site, the surface hardpan was not as dense; therefore, the relatively clean loose sands were densified using deep dynamic compaction. A 29,000 kg weight was dropped by freefall from a height of 30.5 meters. Primary drops were located on a 10.7 meter grid, and secondary drops were made at the center of the grid. As the groundwater table was only 1.2 meters below the surface, dewatering was needed to bring the groundwater to 3.4 meters below grade in order to carry out the work.

The pre-performance testing at the 5,250 square meter Missile Assembly Building verified that there was a surface loosening with some limited zones of soil not meeting specifications to the 9.1 meter depth; therefore, the upper zone was densified with deep dynamic compaction and the lower zone was densified by compaction grouting.

The treatment method for each of the structures is indicated in Table 2. Four thousand meters of cone penetration tests, 1,000 meters of dilatometer testing and 76 meters of standard penetration tests were used to verify the specified results. Each method was successful in achieving the required improvement; i.e., relative densities of 65 to 70 percent in the clean sand and dense columns of stone or compaction grout to reinforce the cohesive layers.

TABLE 2

Summary of Deep Soil Improvement Information Data
Trident Submarine Base, Kings Bay, Georgia

Structure	Footprint Area (m^2)	Improvement Depth (m)	Number of Compaction Points	Treatment Method
Reentry Body Complex (RBC)	16,192	14.6	2,100	SC
Vertical Missile Packaging Building-2 (VMPB-2)	4,025	13.1	487	SC
Missile Inspection Building (MIB)	4,638	12.8	604	SC
Motor Transfer Facility (MTF)	3,550	14.3	452	CG
Motor Assembly Building-2 (MAB-2)	5,252	13.7	690	DDC/CG
Radiographic Inspection Building (RIB)	6,098	14.0	51/49	DDC
Missile Motor Magazines				
Phase I	5,019	9.1	700	CG
	12,046	9.1	1,680	SC
Phase II	7,027	9.1	980	CG
	11,043	9.1	1,540	SC

SC - Stone Columns
CG - Compaction Grouting
DDC - Deep Dynamic Compaction

CONCLUSION

Efficient and economical solutions to problems caused by poor soil conditions require a thorough evaluation of initial conditions, project needs, method capabilities, and, in most cases, a field test program. Four projects have been described in which two or more methods have been used successfully to accomplish the required ground improvement. In each case the special advantages and limitations of the methods were matched with unique project requirements. As the need to develop poor sites continues to expand and the techniques for ground improvement continue to develop, it is likely that more and more projects will encompass overall soil improvement by combining methods.

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