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SYNOPSIS Since its development as a full-fledged soil compaction technique by Louis Menard in the late 1960s, the Dynamic Compaction method has evolved considerably and has become an increasingly efficient ground improvement system.

Among the special techniques borne out of this evolution is the use of Dynamic Compaction to create large-diameter columns using select granular material. This method serves to not only provide increased support and better distribution of imposed loads through the columns themselves, but also augments the capability of the process in improving the host soils to a depth and degree not possible using conventional DC methods.

Three case histories are presented to illustrate this technique and its benefits in terms of increased effectiveness and range of application.

INTRODUCTION

Since its early development in the late 1960s, when Dynamic Compaction (DC) simply consisted of impacting the soil surface with a small compaction plant in a single pass, the technique has benefited from many developments which have increased its effectiveness and range of application. The most notable of these developments are the application of energy following specific geometric patterns that are determined according to the depth and degree of improvement required, the staging of treatment so that compactive energy is applied in several phases, and the development of special rigging and equipment to deliver ever greater energy per drop.

The use of DC methods to construct large-diameter columns with select granular materials is the subject of this paper. The technique is described, its potential applications are discussed, and three case histories are given to demonstrate how the technique has been applied to meet a variety of soil conditions and foundation requirements.

The first case history demonstrates the enhanced improvement achieved through the use of select-fill columns in alluvial soils at an Exshaw, Alberta, cement plant. The second case history concerns the improvement of clay fill using displacement columns and locally abundant bottom-ash as select-fill at a coal mine in Wabamum, Alberta. The last case history discusses a project in Trois-Rivieres, Quebec, where sand columns were used to treat a site underlain by loose saturated fine alluvial soils.

THE SELECT-FILL DISPLACEMENT TECHNIQUE

As DC methods developed, it was recognized that deep, large-volume craters could be made by applying high-impact energy to widely-spaced compaction points. Filling these craters with select material created granular columns with superior foundation properties to those of the host soils. This concept was initially used only as a means to extend the application of DC to sites underlain by fine-grained and organic soils deemed difficult to improve using conventional methods.

In more recent applications, DC select-fill columns have been employed to support heavy point loads, transfer loads from surface foundations to more competent soils horizons at depth, evenly distribute loads under heavily-loaded slabs or embankments, improve slope stability (Kruger, Guyot, Morizot, 1980), or simply enhance vertical drainage. This modified method involves treating specific widely-spaced compaction points with above-normal compactive effort or "hyper-compaction" to form deep craters which are then backfilled with select-fill material. These backfilled compaction points may then be re-compacted and re-filled with additional select material to achieve adequate displacement and volume reduction of the host soil. Additional phases of treatment and column formation are then carried out at intermediate points until the desired level of overall improvement is attained. In view of the applications and methods used, it can be said that this technique has much in common with vibro-replacement applications. The most significant difference, other than the wider range of soils that can be treated with DC, is that DC columns can achieve considerably greater improvement of the host soils within its column depth range, while vibro-replacement is potentially capable of greater column and influence depths.

Select-fill columns can be formed by direct displacement, by pre-excavation and replacement, or by a combination of both methods. The select backfill creates very dense, high-friction angle charges of material that advance as a unified granular mass. As the columns are pushed into the ground, their vertical and radial expansion results in densification of the host soils to a level not possible using conventional DC methods. The granular columns also greatly improve the transmission of DC energy to greater depths than is feasible with conventional treatment. Applications have shown that the depth of improvement predicted by:

$$d=\alpha\sqrt{MxH} \tag{1}$$

where d is the improvement depth in metres, α is the depth efficiency coefficient, M is the tamper mass in tonnes, and H is the tamper drop height in metres, can be significantly increased using select-fill displacement methods. Using standard compaction equipment, columns 3 to 4 metres in diameter and up to 7 metres in depth can be formed. Larger columns can be made by excavating and replacing prior to compaction and/or by increasing the energy per impact. The select-fill is usually clean granular material from the most economic available source.

Recent applications in saturated soils of silt, clay and peat composition suggest that column formation can result in soil mixing significant enough that the host soil is transformed into a composite of its original constituent soils and the select-fil column materials. Lo, Ooi, and Lee (1990) have described a technique of soil mixing that they call Dynamic Replacement, which, based on their findings, can achieve a relatively uniform dispersion of the select-fill materials into the host soils.

ESTIMATING COLUMN DEPTH

The application of the column formation technique remains a subject of much empiricism. The penetration depth of the tamper, which is obviously a major consideration, is not easily predicted. Based on a simple rule formulated by Menard in 1973, the penetration of the tamper increases with the log of the number of impacts according to the following formula:

$$p=\beta\log n \tag{2}$$

where
- p = the penetration of the tamper in metres,
- n = the number of drops of a tamper, and
- β = a coefficient which is a function of the nature and density of the soil, the geometry of the tamper, the energy upon impact, and the fill material.

Table 1 gives the values of β that were obtained experimentally when constructing displacement columns in various types of soils using tampers having a base area in the order of 3 square metres and delivering an energy-per-drop of between 400 and 500 tonne-metres.

Chow and al (1992) have developed a simplified model based on the one-dimensional wave equation, which has been shown to provide computed penetration values that are in good agreement with field measurements from two Dynamic Compaction projects, and which may eventually develop as a valuable design tool.

TABLE 1 -- Observed Values for β

Material	β
Very loose municipal landfill	3 to 4
Medium loose to medium dense sand	1.8 to 1.4
Loose blasted rock-fill	1.6
Roller-compacted rock-fill	0.85

It must be emphasized that the addition of select material during DC operations increases the efficiency of crater expansion in the host soil, which may have a significant effect on the value of β.

CEMENT SILOS, EXSHAW ALBERTA

In 1988, Lafarge Canada Inc. undertook to expand their cement storage and loading capacity at their Exshaw plant in Alberta by constructing a new four-silo cement storage complex. DC was chosen to provide adequate ground improvement to allow construction of the silos on a raft foundation. This case history describes the use of select-fill columns in satisfying the foundation requirements.

Site conditions

The plant site is located on a predominantly granular alluvial terrace on the north bank of the Bow River. The silo foundation soils consist of loose to compact gravel, sand and silty soils to a depth of 12.5 metres, followed by dense gravel and sand layers extending to the limit of the borings at 38-metre depth. From previous soil investigations, it was known that competent strata extended to a 60-metre depth. Ground-water was encountered at an 18-metre depth.

In preparation for Dynamic Compaction treatment, a 3-metre thick surface layer of silt was excavated from the silo foundation area and replaced with the same 50mm minus "select fill" (crushed limestone) material designated for use as backfill during DC operations. The soil profile after excavation and replacement is described in Table 2.

Loading conditions

The silo complex consists of a group of 4 silos with an interstice compartment at the centre, as shown in Fig 1. Each silo measures 10.4 metres inside diameter and stands 68 metres in height. A 1.8-metre thick 27 metres by 27 metres square raft foundation with truncated corners distributes the silo loads over a 724 square metres bearing area. The live-load component represents approximately 83 percent of the total load when all silos are full. Under the maximum total design load of 372,610kN (including the weight of the foundation), the bearing stress is 516kPa.

TABLE 2 -- Soil Profile

Depth (m)	Soil Unit	Description
0.0 - 3.0	A	Crushed limestone, 75% base area; sandy gravel, 25% base area.
3.0 - 7.5	B	Mainly sand, silty fine sand and silt, some gravel. Before treatment, this layer was generally loose.
7.5 - 9.5	C	Mainly gravel. Before treatment, this layer was medium dense to very dense.
9.5 -12.5	D	Mainly fine sand, some silt. Before treatment, this layer was loose to medium dense.
12.5-17.0	E	Mainly gravel. Very dense.
17.0-38.0	E'	Gravel becoming sand. Medium dense.
38.0-50.0	F	Clay. Probably very stiff.

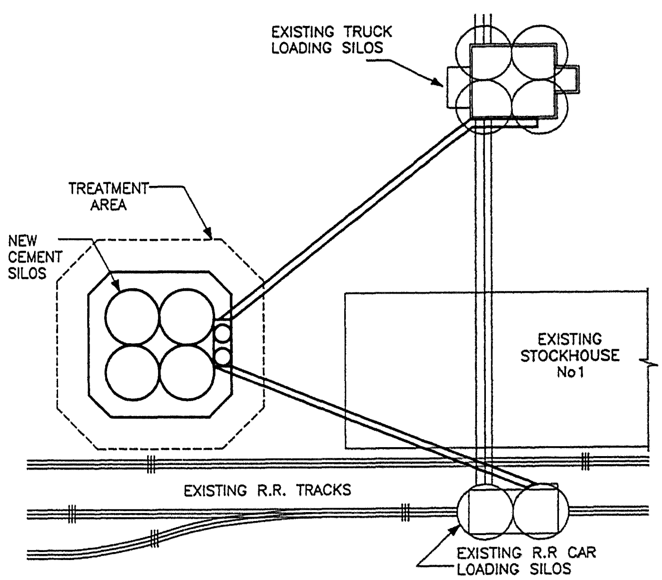

FIG. 1. General arrangement

As the new silos were to be joined by galleries with two existing groups of silos settlement had to be limited so as to minimize movement at the structural connections. The foundation performance criteria established by the structural designer required that total settlement under full load not exceed 75mm and that differential settlement under any loading condition not exceed 38mm, representing an angular deflection of 1 in 725.

Foundation selection

The foundation options considered for this project are compared in Table 3 in terms of cost and schedule.

TABLE 3 -- Foundation Options

Option	Cost	Schedule weeks
Expanded base Compacto piles	$275,000	12 to 14
Jet-grouted Piles/Soil reinforcement	$300,000	12 to 14
Dynamic compaction	$145,000	2 to 3
Excavate and replace loose soil stratum	$250,000	4 to 5

Dynamic Compaction was chosen as the most effective approach in terms of both technical capability and economic benefits. Schedule was a particularly important consideration, since the contract, calling for completion of the silos by April, 1989, was awarded on August 12, 1988.

Development of the compaction program

Existing adjacent silos were founded on expanded-base piles that extended to the top of the soil Unit E (see Table 2) at a depth of 12.5 metres. Based on the satisfactory performance of these structures, it was concluded that adequate improvement of soil units A, B, C and D would provide satisfactory foundation performance for the new silos.

Because of the magnitude of the bearing stresses, the variable live-load distribution and the demanding settlement criteria, conventional DC procedures were not considered adequate for the prevailing soil conditions. The proposed approach consisted of a combination of very high-energy or "hyper-compaction" and select-fill displacement. The uniqueness of this approach can be appreciated by comparing the amount of energy applied on this project with that of other projects. Whereas conventional DC applications rarely exceed 5,000 tonne-metres of energy per compaction point, this silo site received up to 65,000 tonne-metres of energy per compaction point.

Execution and control of the ground improvement work

The compaction was carried out using a 17-tonne steel tamper applying 425 tonne-metres of energy per impact. In addition to the initial site investigation testing, Pressuremeter Testing (PMT) was carried out before treatment and PMT, Standard Penetration Testing (SPT), and Becker Hammer Drill Testing (BDT) were carried out during and after treatment to control the work and to assess the improvement.

Phases 1 & 2 compaction were carried out on a 12-metre grid with Phase 2 compaction points at mid-points between the Phase 1 locations. Phases 1 & 2 employed very high energy in conjunction with select-fill displacement. Phases 3 & 4 were applied between compaction points of previous phases at 8.5 and 6-metre spacing on-centre respectively. These phases were carried out in a single pass using conventional methods but with a high-level of energy to achieve the degree of densification required. A low-energy ironing pass of contiguous impacts from a reduced drop height completed the compaction program.

The change in the rate of volume increase of compaction craters was taken as the indicator of the stiffness of the foundation soil and the degree of improvement achieved. Fig. 2 gives a pair of typical crater depth and crater volume curves for Phase 1, Pass 1 compaction and shows that the rate of crater volume increase remained significant after 60 impacts, prompting the application of additional Pass 2 compaction until volume changes became negligible at approximately 120 impacts. Fig. 3 shows a consistent depth versus number of impact relationship for all Phase 1 compaction points, demonstrating a uniform response to treatment and providing a measure of confidence in the uniformity of the soil profile and soil consistency across the foundation area.

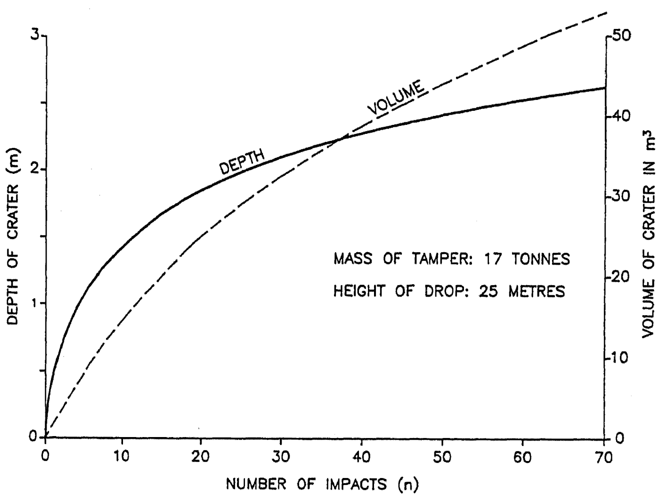

FIG. 2. Crater depth and volume vs number of impacts

The total induced settlement was determined by measuring the change in the before and after elevation of the working platform then adding the volume of crushed limestone driven into the native host soils. Some 2,840 tonnes (3,120 tons) of crushed stone was required to backfill the craters and form the select columns. Of this amount, 1,510 tonnes (1,660 tons) were added within the silo foundation footprint, with the balance applied to the 6-metre wide peripheral compaction zone encompassed the raft. The total induced settlement, accounting for the lowering of the work platform and the volume of stone injected into the host soil, averaged some 1.5 metres, representing a 9.4 percent volume reduction, based on a 16m improvement depth. This level of volume reduction is well above the 5 to 6 percent range typical of normally consolidated alluvial soils.

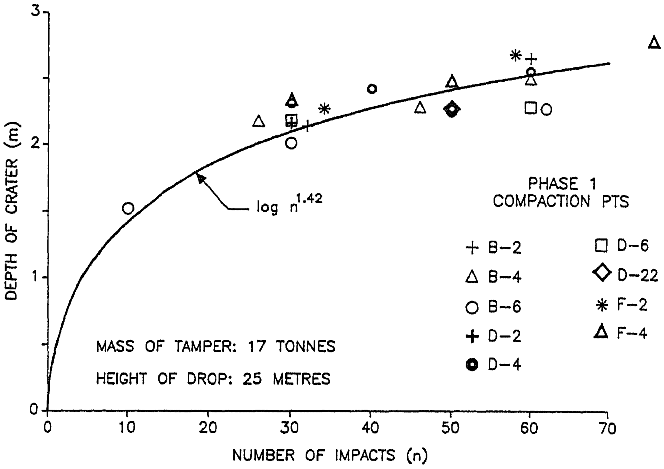

FIG. 3. Crater depth vs number of impacts

BDT testholes located at the centre of hyper-compaction points showed that the select-fill columns extended to depth of over 6 metres.

Final verification testing

Final verification testing consisted of PMT, BDT, and SPT. The very high densities achieved in the coarser granular soil units restricted the application of PMT and SPT testing to the finer-grained soil units B & D. These tests were performed just ahead of an open-ended Becker casing, which was pre-drilled through the coarse stone and gravels of soil units A and C.

PMT testing was the primary verification test method and provided a basis for a load deformation assessment of the improved soils. A total of 22 pressuremeter tests were performed at four borehole locations mid-way between compaction points, with one borehole in each quadrant of the raft foundation.

The Becker Hammer Drill proved effective in penetrating even the coarsest and densest soil layers. Used as a secondary method of verification testing, BDT results provided valuable insight into the extent of the improvement achieved. A comparison of before and after open-ended BDT penetration resistance in Fig. 4 shows consistent and substantial improvement over the full 16-metre test depth. Based on formula (1), the 16-metre improvement depth represents an above-average depth efficiency coefficient (α) of 0.78. An unexpected level of improvement was achieved in Unit E soils at a depth of 16 metres, where initial BDT penetration resistances of approximately 100 blows were doubled. This data provides an excellent demonstration of the enhanced capabilities of the hyper-compaction/select-fill displacement approach in terms of both depth and magnitude of improvement.

Side by side PMT and BDT test results were

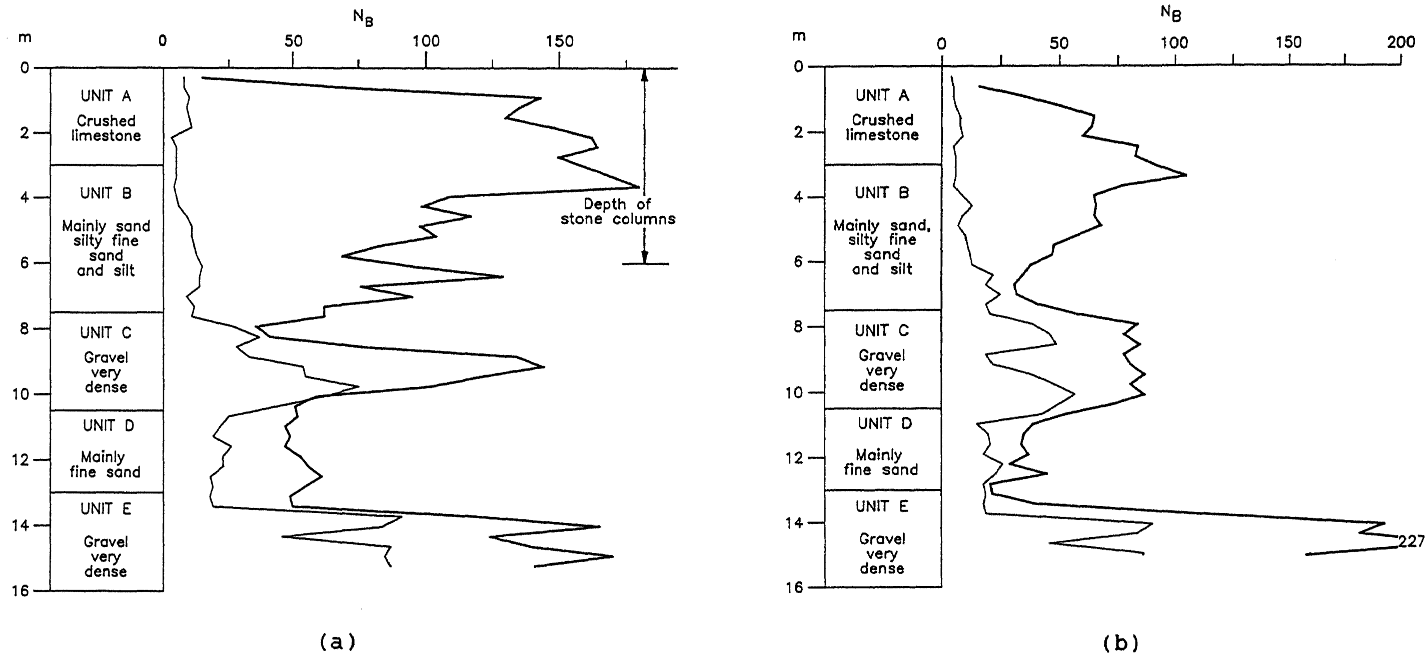

FIG. 4. Before and after open-ended BDT comparison, (a) on center of a phase 1 crater, (b) average of all tests

found to correlate well, and, using this correlation to augment the PMT data, calculations for bearing capacity and settlement were performed in conformity with Menard's empirical pressuremeter methods, as described in Notice D/60/67 of the Centre d'Etudes Geotechnique (1967).

The results of all calculations are summarized in Table 4. The calculated bearing capacity and settlement values, together with PMT and BDT test results, indicated that the objectives of the ground improvement program had been satisfied.

TABLE 4 -- Summary of Pressuremeter Calculations

	PMT-1	PMT-2	PMT-3	PMT-4
Ea, bars[1]	347	342	401	325
Eb, bars[1]	194	198	198	196
Settlement, mm	41	42	37	43
Bearing capacity, kPa	660	660	670	670

1 Ea AND Eb are the PMT moduli corresponding to the zones of the spherical and deviatoric tensors respectively.

Post construction performance

Post construction settlements were monitored over a period of 580 days following completion of the silo slip-forming. The measurements were made on steel bolts anchored into the reinforced concrete wall of each silo, providing for settlement determinations at the four corners of the foundation.

The results of the settlement surveys are presented graphically in Fig. 5, which shows the relationship between load, settlement and time. The maximum total settlement of 55mm represents an average of the four readings under full load and is well within the specified post-construction limit of 75mm. The maximum measured differential settlement is 3mm, representing an angular deflection of 1 in 9,000.

The measured total settlement exceeds the predicted pressuremeter settlement by approximately 32 percent. The difference between predicted and actual settlement is expected to be related to an over-estimation of modulus values derived from BDT correlations below the PMT test depth and/or higher-than-actual modulus values assumed for soils below test depth. In this regard, the thick stratum of stiff clay that occurred at depth may have contributed more settlement than expected.

MINE SERVICES BUILDING, WABAMUN, ALBERTA

The Highvale Mine lies on the shore of Lake Wabamun, approximately 80 kilometres west of Edmonton, Alberta. In 1981, Highvale undertook construction of a new mine service building on a site underlain by over-burden mine spoil from the coal-mine operations. The spoil was in a loose condition and required improvement to provide adequate foundation support for the proposed structure.

The following case history describes the DC

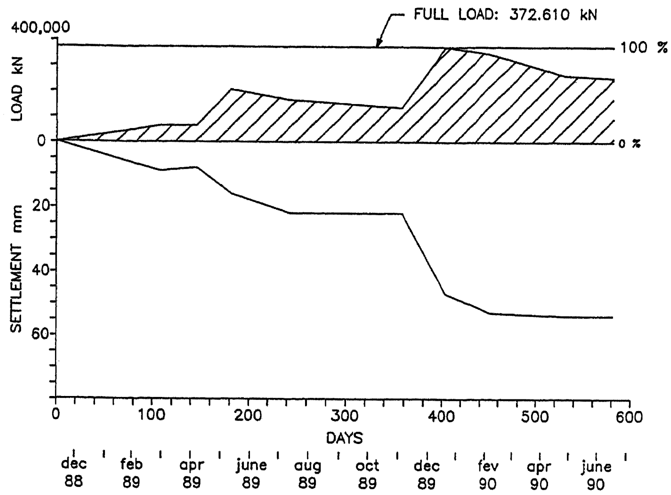

FIG. 5. Load/settlement diagram

select-fill displacement methods used in treating the clay-fill mine spoil. A total of 24,000 cubic metres of bottom-ash material was needed to construct the columns. The suitability of bottom-ash waste from the adjacent Sundance Thermal Generating Plant as select-fill material represented an additional economic benefit over and above already attractive ground-improvement costs.

Site Condition

The subject site was mined out and backfilled with spoil materials in the period from 1975 to 1976. The mine spoils are a heterogeneous mixture of stiff clay lumps mixed with broken pieces of shale, sandstone, siltstone and coal with occasional lenses of silty sand. The spoil extended to a depth of 14 metres and overlay intact coal and shale strata. The water table was at a depth of about 8 metres.

The site investigations which preceded construction showed that N values in the spoil ranged from 3 to 40. Continuous dynamic cone soundings done in parallel with the SPT testing revealed the occurence of numerous thin intervals of low penetration values which, when investigated by means of test pitting, were shown to correspond to voids and inclusions of loose material between stiff lumps of clay.

Laboratory tests performed on spoil samples showed that its clay fraction varied between 14 and 70 percent, its liquid limit between 48 and 75 percent, and its plasticity index between 19 and 51 percent. All but two of the 16 samples tested lay just above the "A" line, indicating inorganic clays of high plasticity.

Loading conditions and foundation selection

The Mine Services building was designed to accomodate offices and heavy equipment maintenance garages. Typical column load in the office and garage are 36 and 68 tonnes respectively. The garage was designed to service coal carriers which, if fully loaded when brought in for repairs, weigh 220 tonnes and exert a bearing pressure of 540kPa under each of the two pairs of rear-axle tires.

The foundation approach proposed by the designer and adopted by Highvale was to treat the mine spoils using DC and support the structure on an engineered fill pad of roller-compacted fly-ash, taking advantage of the lightness and self-cementing properties of this material to better distribute the structural loads to the improved underlying spoil materials. As with the coarser bottom-ash, the fly ash material was a readily-available by-product of the adjacent thermal generating station.

Development of the compaction program

Fills of clayey composition can be densified by DC, but such treatment can be complicated by high excess pore pressures and plastic deformation around impact points. The ground improvement plan consisted of densifying the spoil materials by creating large-diameter columns of select-fill material to displace the host soils both radially and vertically. The compaction design called for the formation of columns in four successive stages. The first stage columns were installed on a 10-metre grid, with those of each successive stage located at the mid-point within the columns of the preceding stages. This procedure was adopted to promote rapid dissipation of local excess porepressure through the granular columns themselves.

The bottom-ash, a by-product of coal combustion, was a sand-size material having the gradation shown in Fig. 6. Its dry unit weight when compacted by vibratory roller ranges from 10.4 to 13.1 kN per cubic metre making it a light fill material. Its use as a backfill material in deep craters formed by DC would provide a dense mass capable of displacing the host soils to effect compaction and improving the transmission of compactive energy to the spoil materials below and around the select-fill columns.

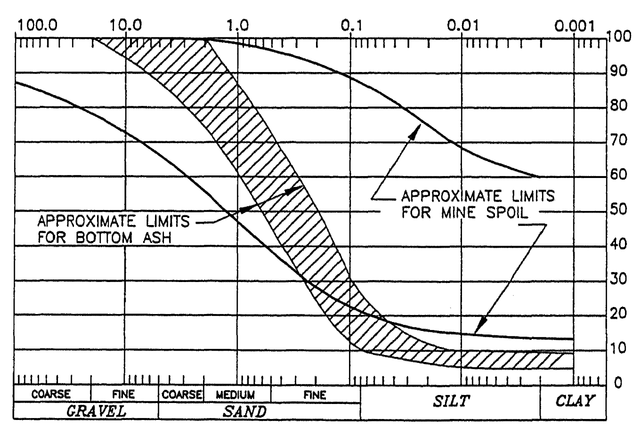

FIG. 6. Gradation limits for bottom-ash and mine spoil

Execution and control of the ground improvement work

The treatment of the 16,600 square metres area was performed over a period of nine weeks using an average 400 tonne-metres of energy per impact for select-fill column formation. A total of 1,267 columns were formed, consuming over 24,000 cubic metres of bottom-ash fill. This corresponds to a volume reduction in the order of 10.3 percent for the spoil material within the building footprint. On the basis of many previous applications, 8 to 10 percent volume reduction in young fill materials indicates an excellent response to treatment.

The compactive energy applied over the foundation area varied from 29 to 32 tonne-metres per cubic metre of treated spoil material. The average depth of Stages 1, 2, 3 and 4 columns was 5.03, 2.80, 2.13 and 1.46 metres respectively. A concentration of 6-metre deep first stage columns was recorded at the south end of the site, and a local area which ponded water before treatment had the deepest columns at just over 7 metres. For the formation of the columns, Fig. 7 shows the rate of column expansion per tonne-metre of energy as a function of total energy applied to the site. As would be expected, the maximum rate of volume reduction occurs with the Stage 1 column formation and reduced thereafter as the host soils were improved incrementally by each successive stage of treatment. This data gives a measure of the densification effect the select-fill displacement columns had on the host soils as the work progressed.

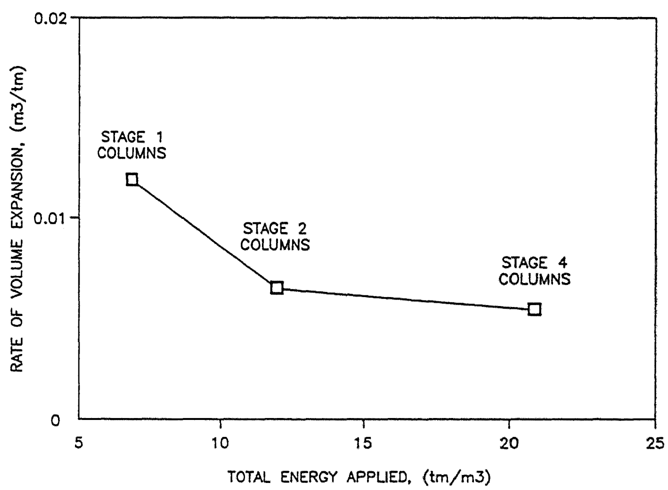

FIG. 7. Change in column expansion as a function of total energy applied

Pore pressures were monitored by means of 18 pneumatic piezometers. During the ground improvement, only two of the piezometers recorded significant sustained excess pore pressure. This response suggests that the granular columns provided effective drainage for the locally high excess pore pressures, which in turn promoted optimum volume reduction of the mine spoil materials.

Verification testing

Final verification testing was carried out using the pressuremeter (PMT), standard penetration (SPT), and dynamic cone penetrating (DCPT) tests. All testing was performed in the spoil material between columns. Table 5 presents a comparison of before and after test values averaged over the full spoil depth. All tests demonstrated an appreciable improvement of the spoil as a result of the treatment and, because tests were carried out between column locations, represent a conservative measure of the after-treatment condition of the overall soil mass.

TABLE 5 -- Comparison of before and after insitu test data

	PMT MODULUS bars	SPT N	DCPT N'
BEFORE	170.0	14.0	19.0
AFTER	257.0	22.0	41.0
IMPROVEMENT FACTOR	1.51	1.57	2.15

Maximum improvement occured at a depth of 4 to 5 metres and remained significant through the full depth of the spoil as shown by the before & after DCPT comparison in Fig. 8. Based on formula (1), the 14-metre improvement depth represents a depth efficiency coefficient (α) of 0.7.

An excellent indication of effectiveness in reducing the potential for differential settlement is the variation index given by:

$$i = \frac{e}{\overline{w}_s} \qquad (3)$$

where
- i is the variation index
- e is the standard deviation of calculated settlement,and
- ws is the mean value of specific settlement.

The value of the variation index for this clay fill soil mass was reduced from 0.58, based on before treatment tests, o 0.26, based on after treatment tests, representing an improvement of over 100 percent.

Post-construction performance

The work was evaluated and approved on the basis of a settlement analysis using pressuremeter data. This analysis assumed that column loads were applied directly over the improved spoil materials and ignored the structural fly-ash raft under the foundation in terms of both additional load and added strength. Calculation gave total settlement values ranging from 4 to 9 mm and the worst case angular deflection to be 1:1575.

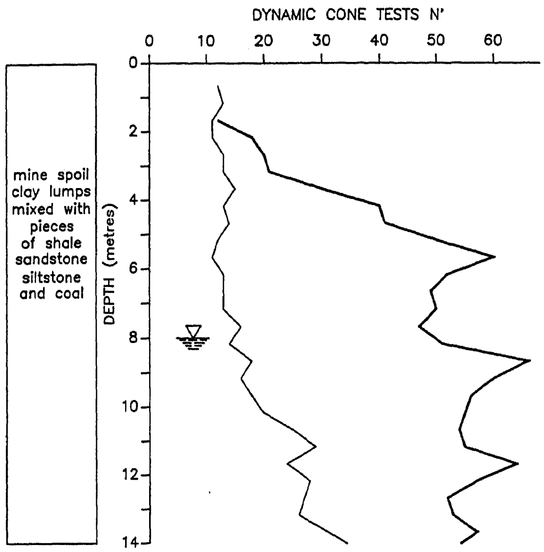

FIG. 8. Average before and after Dynamic Cone Penetration Resistance values between columns

Wade (1983) reported that the maximum total settlement measured 6 months after completion of the building was 20mm and the maximum angular deflection was 1:4,000. A re-calculation of settlement using the same PMT data as previous but adding the load represented by the 3-metre thick structural fly-ash raft placed following Dynamic Compaction gives a predicted settlement of 19mm, which is in excellent agreement with observed performance.

PRICE CLUB WHOLESALE STORE/WAREHOUSE, TROIS-RIVIERES, QUEBEC

In 1992, construction of a Price Club Wholesale Store/Warehouse was undertaken in Trois-Rivieres, Quebec. Foundation conditions on the site were such that improvement was called for in order to allow construction to proceed with conventional spread footings and slab-on-grade floor. This case history describes the Dynamic Compaction program of treatment on saturated fine-grained alluvial soils. Select-fill displacement columns were created in the upper 4 to 6 metres and the monitoring of pore-pressure response was critical to the effectiveness of the soil improvement program.

Site condition

The site is situated at the confluence of the St. Maurice and St. Lawrence Rivers, on the north shore of the St. Lawrence. It is underlain by a sequence of deltaïc deposits to a depth of 26 metres, followed by a 5-metre stratum of Leda clay and a dense glacial moraine extending to an undertermined depth. The groundwater table lies just 0.1 metres below surface.

Table 6 describes the soil sequence following removal of the thin topsoil cover and the placement of a 2.3-metre clean sand fill pad designed to provide confinement of the native soils during treatment and to ensure a competent free-draining work surface for the compaction operations.

TABLE 6 -- Soil Profile

Depth (m)		Description
0.0-2.3	SW	Fill, well graded sand.
2.3-6.5	ML	Silt with betweem 20 and 30% fine sand, $1 \leq N \leq 2$
6.5-11	ML-SM	Silt and fine sand, $2 \leq N \leq 7$
11-30	SM	Fine silty sand becoming medium, $N \geq 11$.
30-26	ML-SM	Compact grey sandy silt.
26-30	CL	Grey clay with trace of silt, soft.
30-33	SP-SM	Dense fine to medium sand with traces of silt.
33		Limit of investigation

Typical gradation curves of the upper ML and ML-SM horizons are shown in Fig. 9. Before treatment water content in the ML horizon decreased from a high of 57 percent near the surface to values ranging from 24 to 27 percent below 1 metre. The plasticity chart of Fig. 10 shows the range of Atterberg limits measured for the ML soils.

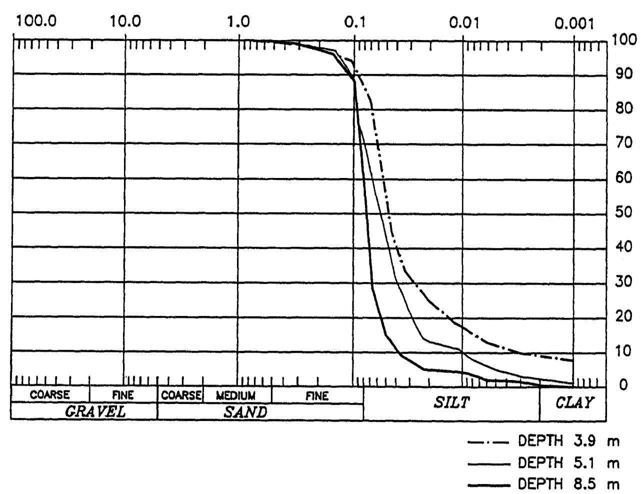

Fig. 9. Typical gradation curve in the upper nine metres of alluvial soils

Loading conditions and foundation selection

The building comprises a single-storey warehouse type structure with an attached two-

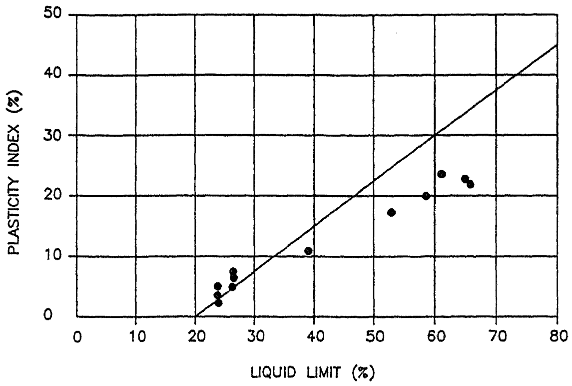

Fig. 10. Plasticity chart showing test data for ML soils

storey office. The fill required to raise the site to grade, together with the structural and live-loads, applied an average bearing stress of 40kPa over the building footprint area of 12,800 square metres. The maximum design columns loads are 800kN. The maximum floor loads of 5,120kN are applied over 8m x 16m rack assemblies. Footings were sized for an allowable bearing pressure of 200kPa.

Two foundation alternatives were considered: a fully structural solution based on piles, grade beams, and heavily reinforced floor slab; and a foundation on conventional footings and slabs-on-grade following ground improvement by Dynamic Compaction methods. The latter solution offered an estimated 1 to 1.2 million dollar economy in addition to reducing the construction schedule by months and was, therefore, preferred.

Development of the compaction program

The main objective of the treatment program was to improve the upper silt and sandy silt soils between 2.3 metres and 11.0 metres depth. The most important component of total load was the thickness of the sand fill required to raise the site to final grade. Except for long-term consolidation in the deep clay horizon, which was not a concern due to its depth below foundation level, consolidation due to the fill load was expected to be largely complete at the time of building construction.

The compaction plan consisted of the formation of columns through the upper silt layer in four to five successive phases of treatment followed by two additional phases using standard DC procedures. A system of horizontal drains was installed in the treatment area prior to DC to intercept water from pore-pressure relief and facilitate its migration away from the treatment area. Pneumatic piezometers were installed to evaluate pore-pressure response and to determine the lag-time required between successive phases of treatment.

Execution and control of the ground improvement work

Formation of sand columns was carried out using a 13.6-tonne tamper and an energy-per-impact of 260 tonne-metres. The first group of columns was installed on a 14-metre square grid and columns were expanded in 4 to 5 successive passes. Detailed survey was employed to ensure that heave represented less than 20 percent of the column volume and sufficient time was allowed between passes for dissipation of excess pore pressures. The second group of columns was formed in between the first, and, similarly, the third group in between the preceding groups. The procedure for expanding the columns was the same for all groups. The formation of columns was followed by two additional phases of low-energy compaction using standard DC procedures.

The energy applied in forming the Group 1,2, & 3 columns was 12,000, 9,1000 and 6,500 tonne-metres respectively. Testing showed that Group 1 columns were between 5 and 5.5 metres deep. The depth of Group 2 & 3 columns was not determined directly but is expected to be in proportion to the depth of Group 1 columns, based on the energy applied. The total energy applied over the entire site was 4 million tonne-metres, representing an average applied energy of 268 tonne-metres per square metre of site. Sixty-seven percent of the energy was applied for column formation and the remainder for conventional final lower-energy compaction and ironing. Menard pressuremeter and Marchetti Dilatometer tests were carried out during treatment to assess the degree of improvement achieved when compared with "before" test data.

A total of 10,800 cubic metres of sand went into the formation of columns. The total volume reduction achieved, taking the depth of improvement to be 10 metres, amounts to roughly 7.2 percent, which is well above the 5 to 6 percent historical average for saturated fine-grained alluvial soils.

As mentioned earlier, the horizontal drainage system was installed just below the fill to intercept the upflow of groundwater. This system, which had zero flow prior to DC, discharged continuously during the 45 days of the compaction work. The volume of water transported by the drainage system was monitored and, because the site was encompassed by diversion ditches and virtually no precipitation was experienced during the work, the flow volumes are known to represent displaced pore water from within the treatment area. Flow measurements indicated that over 5,000 cubic metres of water was handled by the system. A typical pore-pressure record is shown in Fig. 11 to demonstrate the effectiveness of controlled column expansion in promoting dissipation of pore pressures.

Final verification testing

The effectiveness of the ground improvement program was assessed by means of 10 PMT and 24 DMT testholes. Fig. 12 shows the average before and after results for DMT & PMT. While the magnitude of improvement varies for the different parameters measured, all tests

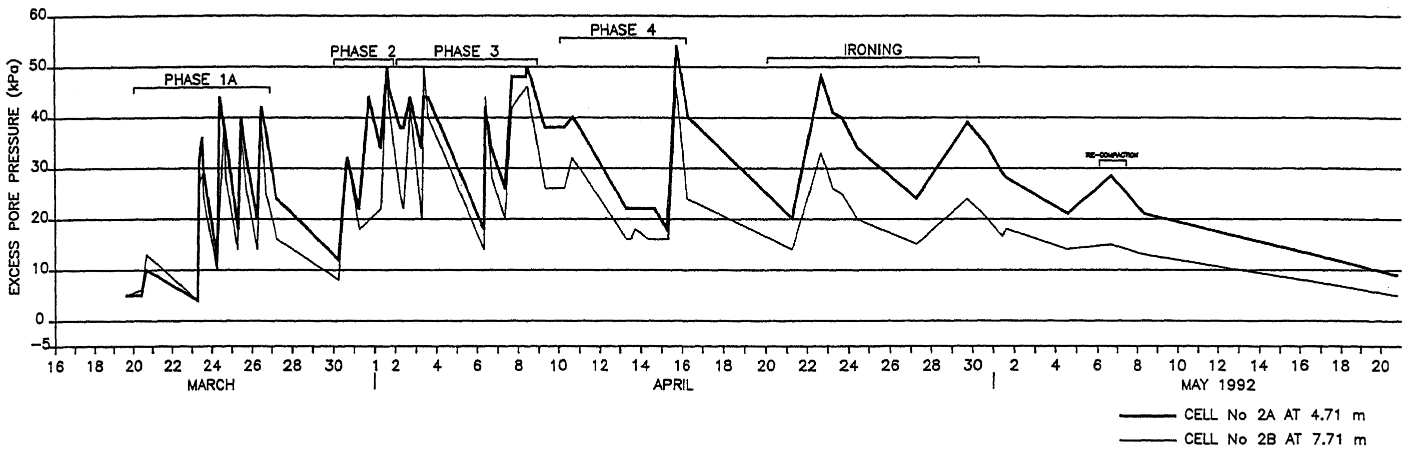

Fig. 11. A typical pore pressure record

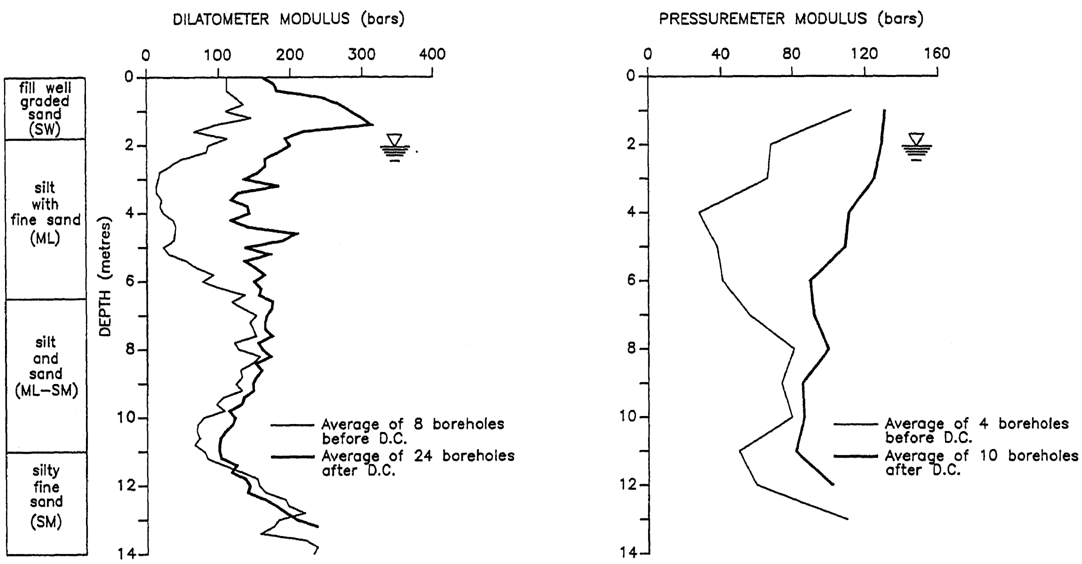

Fig. 12. Before and after comparisons of DMT and PMT moduli

demonstrate consistent improvement to depths of between 10 and 12 metres. Based on formula (1), the 10-metre improvement depth represents an efficiency coefficient (α) of 0.6, significantly greater than values of 0.35 to 0.40 suggested by Lukas (1992) for saturated semi-pervious soil. Figure 13 shows average before and after DMT material index profiles, with after profiles given for tests carried out on columns and between columns. It is noteworthy that the change in material index is similar for both after-treatment profiles, suggesting that significant soil mixing has occurred within the depth of influence of the sand columns.

Post-treatment pressuremeter test data was used to assess foundation performance in terms of bearing capacity and settlement. Predicted total settlement varied from a low of 5mm to a high of 10mm for a confirmed allowable bearing stress of 200kPa. At the time of writing this paper, construction of the building was in progress and, therefore, no foundation performance data was yet available.

CONCLUSION

It is shown that the introduction of select granular fill and the application of above-average compactive effort enhances Dynamic Compaction improvement. It is suggested that the two main factors influencing the observed increase in inprovement are a) an increase in the efficiency of crater volume expansion, which increases volume reduction in the host soils and b) increase in the efficiency of energy transmission from the impacting tamper to the surrounding host soils, which enables more of the energy to effect mechanical rearranging and/or increases in pore pressure.

Comparisons of before and after test results show that improvement depths are significantly greater using select-fill displacement methods

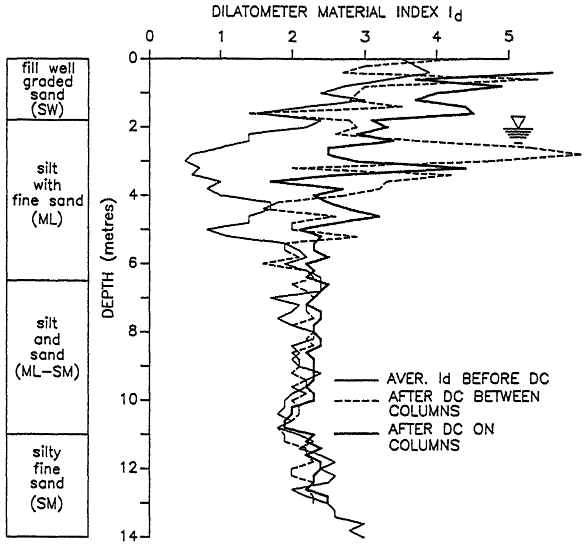

Fig. 13. DMT Material Index, before and after

than for those predicted by Lukas (1992). It is also demonstrated that the degree of overall improvement is significantly enhanced over what would be achieved using conventional methods. Before and after DMT material index profiles indicate that significant soil mixing occurs in saturated fine-grained alluvial soils.

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