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Upgrading of Existing Landfills by Dynamic Compaction - A Geotechnical Aspect

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UPGRADING OF EXISTING LANDFILLS BY DYNAMIC CONSOLIDATION A GEOTECHNICAL ASPECT

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ABSTRACT: In recent years, the scarcity of land space available for new urban development has prompted a renewed interest from local authorities in the end use of various landfills or in the extension of the life of existing landfills. Rehabilitation of closed landfills for urban developments has received considerable interest. Likewise, the extension of landfill life to allow for more waste storage is also receiving equal attention. In both cases, ground improvement is required.

Dynamic consolidation (also known as dynamic compaction) is a ground improvement technique. The process involves dropping heavy weights (15ton - 20tons) on to the surface of the fill from a considerable height (15m - 20m) following a selected grid pattern. These high-energy impacts produce sufficient compaction effort to reduce void space, increase density and reduce long-term settlement of the fill. By increasing the density, it increases the storage capacity of the landfill. Beside, it also increases the bearing capacity. Reducing the long-term settlement, roads, parking bays and lighter structures can be designed on shallow foundations on closed landfills.

In this paper, the subject of settlement of waste fills is addressed. A case study concerning a housing development over a landfill is also presented.

1.0 INTRODUCTION

Landfilling is one of the most economic and feasible means of disposing municipal solid waste in Malaysia and other countries in Southeast Asia. In the past, the disposal of waste fills was carried out by uncontrolled dumping into ex-mining ponds and low-lying areas close to housing estates. With increasing scarcity of land in urban areas, it is increasingly difficult to find new landfill sites for future dumping. This has prompted the local authorities and privatized companies (operators of landfill) to find solution to extend the life of the landfill to allow for more waste storage.

Typical landfills may occupy an area ranging from several acres to hundreds of acres. Settlement estimation is a topic of concern. From the operator's viewpoint, landfill capacity will be increased if most settlement occurs during the stage of filling. Unfortunately, the landfill settlement continues over an extended period of time with a final settlement that can be as large as 30%-40% of the initial fill height (H.I.Ling, et.al. 1998). Hence, it is imperative that a solution is needed to increase the rate of settlement to recover the additional space.

Dynamic consolidation is a good method of compacting refuse and waste fill. This technique involves dropping heavy weights (15 – 20 tons) on to the surface of the fill from a height of 10 to 20m following a selected grid pattern. The high-energy impacts produce shock waves that propagate to great depths (figure 1). As a result, the density of the waste fill is increased and hence, the storage capacity of the landfill is also increased.

With the increase in the density of the waste fill, the overall bearing capacity is improved. The long-term settlement is reduced and hence, the differential settlement is also reduced which is important for the integrity of the cover system when the landfill is closed. In the past such landfills have been considered suitable only for green areas. With the increasing scarcity of land in urban areas, it is making it necessary to build structures above such fills. Charles et.al. (1981) report several case histories of construction on old refuse tips, which include construction of a 2-storey hospital, roads and highways. Welsh (1983) cites a roadway site with 6m to 12m of waste fills. Mènard (1984) cites a case for a warehouse designed with floor loads of 20 kN/m^2 and spread footing with 145 kN/m^2 with 6m to 17m of refuse waste. There are many other recorded and published case studies on such developments (e.g. Aziz & Mohd. Raihan (992), Downie & Treharne (1979), Faisal, K.Yee & Varaksin (1997), Fryman & Baker (1987), Lewis & Langer (1994), Mappleback & Fraser (1993), Steinberg & Lukas (1984), etc.).

In this paper, the subject of settlement of waste fills and rehabilitation of landfill for housing development is presented. Only the geotechnical aspect is covered. The related environmental issue has been intentionally left out due to space constraint.

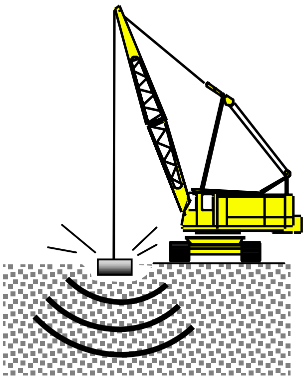


Figure 1

2.0 COMPOSITION OF LANDFILL

Most landfills are heterogeneous and they exhibit anisotropic material properties that are difficult to characterize. Typically, a landfill consists of food and garden wastes, paper products, plastics and rubber, textiles, wood, ashes and the soils used as cover material. Table 1 shows the various components of waste fills with their range of unit weights. The unit weight and void ratio vary with the types of waste, composition, depth, method of compaction and the rate of decomposition, among other factors. The rate of decomposition is further complicated by several factors including the effects of time, temperature and environmental conditions. In short, it is a combination of all of the problems of soft clay, uncompacted fill, organic consolidation and decomposition and even collapse of cavities and erosion of soil into cavities. It is as heterogeneous as the modern industrial-urban complex that produces it. Hence, its composition varies from community to community and from nation to nation. Thus, the waste properties can be considered as site-specific.

Two different forms of landfill can be defined. The uncontrolled dump is of random composition, dumped loosely from trucks, accumulated without control or compaction, and sometimes covered with a thin layer of soil when it reached its capacity (see figure 2a). At the other end, it is the well-managed sanitary landfill. The materials are spread in layers and compacted by bulldozers and compactors. In some cases, certain wastes such as tires are segregated from others (see figure 2b). Most of the old landfills are the uncontrolled dumps. Until recently, through privatization scheme the landfill operation follows the engineered landfill scheme. Thus, it is expected that developments over old landfills will require more engineering effort.

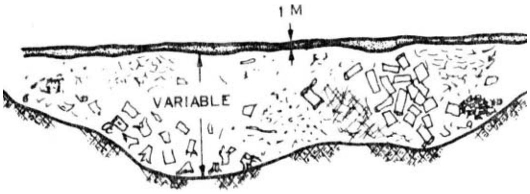


Figure 2(a)
Uncontrolled Landfill (No controlled placement and no compaction)

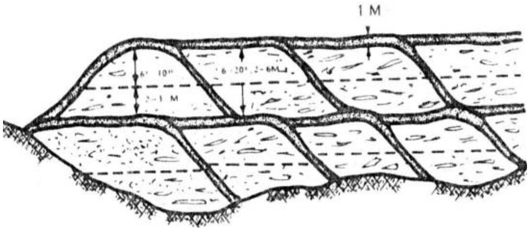


Figure 2(b)
Controlled Sanitary Landfill (Spread and compacted in layers of 2-3m thick; encapsulated with soil in cells of 2-6m thick)

3.0 SETTLEMENT CHARACTERISTICS

Settlement is the major problem with landfills. Sowers (1972) cites a case of a small shopping center built over a landfill. The buildings are on piles driven through the waste fills. The building walls and roof have remained intact. However, floor slabs supported directly on the fill surface have settled as much as 75mm. The floor slabs were connected to the pile-supported exterior grade beams, but was not connected to the interior columns. As a result, the floor drapes downward from the exterior walls toward the interior of the building. Small interior partitions resting directly on the floor have cracked badly and doorframes have been wrecked out of shape.

Table 1 (source: After Tchobanoglous et. al. 1977)

Waste Component	Uncompacted Unit Weight (kN/m^3)	Water Content	Ratio of Compacted to Uncompacted Unit Weight	
			Normal Compaction	Well Compacted
Food waste	1.3 – 4.7	50 – 80	2.9	3.0
Paper / paper board	0.3 – 1.3	4 – 10	4.5	6.2
Plastics	0.3 – 1.3	1 – 4	6.7	10
Textiles	0.3 – 0.9	6 – 15	5.6	6.7
Rubber and leather	0.9 – 2.5	1 – 12	3.3	3.3
Yard waste	0.6 – 2.2	30 – 80	4	5
Wood	1.3 – 3.1	15 – 40	3.3	3.3
Glass	1.6 – 4.7	1 – 4	1.7	2.5
Metals	0.5 – 11.0	2 – 6	4.3	5.3
Ash, brick, dirt	3.1 – 9.4	6 - 12	1.2	1.3

Furthermore, settlement has increased since then, probably due to a change in the moisture environment from leaking sewers in the fill.

There are two possible approaches to the assessment of settlement:

(a) Extrapolation of monitored data obtained specifically for the given fill
  1) By graphical method
  2) By analytical method

(b) Estimation from existing published data on similar type of fills
  1) By graphical method
  2) By analytical method

Method (a) is the most reliable but requires time for monitoring. This method relies on the approximately linear relationship between settlement and logarithm of time elapsed since placement of waste fill. Method (b) relies on published data for other fills of similar type, and gives approximate answers quickly. However, the results are less dependable since the published data are rarely likely to apply exactly to a specific given fill. Preliminary estimates obtained by method (b) should be checked by monitoring. We shall address the different categories of settlement as follow:

3.1 Settlement Under Self-Weight

One of the contributing factors to the overall settlement is caused by the self-weight of the fill. The time-settlement relationship under self-weight is analogous to the secondary compression of soils after a short period of pseudo-primary settlement, typically, 1 to 4 months long. Measurements taken from past records indicate a coefficient of secondary compression ranging from 0.1 to 0.4 (NAVFAC, 1983). Thus, settlement of the waste fills under its self-weight after completion of filling can be estimated by equation (1) below.

$$(\Delta H)_{sw} = H\, C_{\alpha} \log (t_2 / t_1) \quad \ldots\ldots\ldots \quad (1)$$

where
$(\Delta H)_{sw}$ = self-weight settlement at time t_2 (m)
H = thickness of waste fill (m)
t_1 = time pseudo-primary settlement to occur after completion of fill (years)
t_2 = time after completion of fill (years)
C_{α} = coefficient of secondary compression

Table 2 below suggests typical self-weight settlements. According to Leach & Goodger (1991), a good compaction can reduce the self-weight settlement potential by between 50% and 75%.

Typical unit weights for municipal waste are summarized in Table 3.

Table 4 below shows the unit weights obtained from various landfills sites.

3.2 Settlement Under External Loads

The time-settlement behavior of an old waste fills under an applied load is analogous to the behavior of peat. As load is placed large primary (mechanical) settlements occur rapidly with little or no pore pressure build up. This is followed by secondary compression, which occurs over a long period of time.

The relation of the imposed stress to settlement can be expressed as follow:

$$(\Delta H)_p = H\, C_r \log (\{\sigma'_o + \Delta\sigma'\} / \sigma'_o) \quad \ldots\ldots. \quad (2)$$

where
$(\Delta H)_p$ = primary (or mechanical) settlement (m)
H = thickness of waste fill (m)
e_o = initial void ratio

Table 2 (Source: Leach & Goodger (1991) – CIRIA Special Publication 78)

Material	Potential Self-Weight Settlement (expressed as % of depth of fill)
Well-compacted, well-graded sand and gravel	0.5
Well-compacted shale and rockfill	0.5
Medium-compacted rockfill	1
Well-compacted clay	0.5
Lightly compacted clay	1.5
Lightly compacted clay placed in deep layers	1 – 2
Nominally compacted opencast backfill	1.2
Uncompacted sand	3.5
Uncompacted (pumped) clay	12
Well-compacted mixed refuse (waste fill)	30
Well-controlled domestic refuse (waste fill) placed in layers and well compacted	10

Table 3

Description	Average Total Unit Weight γ_T (kN/m^3)	Source
Sanitary Landfill		
• Poor compaction	2.8 – 4.7	Tchobanoglous et.al. (1977)
• Moderate to good compaction	4.7 – 7.1	
• Good to excellent compaction	7.1 – 9.4	
• Baled waste	5.5 – 10.5	
• Shredded and compacted	6.4 – 10.5	
• In situ density	5.5 – 6.9	
Active landfill with leachate mound	6.6	
Household Trash Can	1.1	NAVFAC (1983)
Delivery Truck	2.4	
Sanitary Landfill		
(a) Not shredded		
• Poor compaction	3.1	
• Good compaction	6.3	
• Best compaction	9.4	
Shredded	8.6	
Sanitary Landfill		
• In a landfill	6.9 – 7.5	NSWMA (1985)
After degradation and settlement	9.9 – 11.0	

Table 4

Landfill Sites	Waste Density (kN/m^3)
Old Klang Road, Kuala Lumpur	7.0
Kelana Jaya, Kuala Lumpur	6.0
Merrylands, Sydney[1]	9.4
Thornleigh, Sydney[1]	8.4
Lucas Heights, Sydney[1]	11.3
Albany, New York[2]	7 – 16
Fayetteville, Arkansas[3]	4.8
Richmond, California[4]	7.2

Note: 1 – data obtained from Hausmann et.al (1993)
2 – data obtained from Gifford et.al. (1992)
Note: This is a construction and demolition debris landfill
3 – data obtained from Welsh (1983)
4 – data obtained from Sharma et.al. (1989)

σ'_o = effective overburden pressure (kN/m^2)
$\Delta\sigma'$ = effective imposed stress (kN/m^2)
C_r = compression ratio (= $C_c/1+e_0$)
C_c = compression index

NAVFAC (1983) reports that the primary compression ratio (C_r) ranges from 0.1 to 0.4. Sowers (1972) reports that the compression index (C_c) is related to the initial void ratio as shown in figure 3. The relation can be expressed as follow:

For fills low in organic matters	$C_c = 0.15e_o$
For fills high in organic matters	$C_c = 0.55e_o$

It is interesting to note that the maximum Cc for peat is about one-third greater than the maximum observed for waste fills.

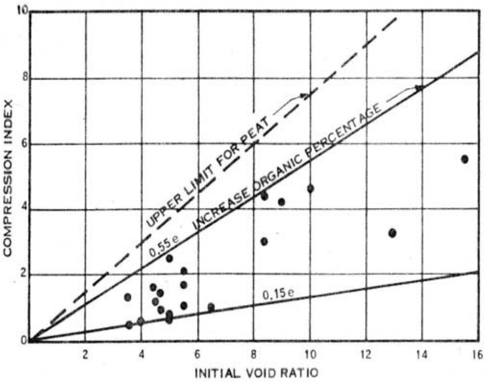


Figure 3

Environmental conditions as well as the composition of the waste fills determine the amount of long-term settlement. This long-term settlement is a combination of mechanical secondary compression, physico-chemical action, and bio-chemical decay. When there is no drastic change in the environment the settlement-log time relationship is more or less linear, similar to secondary compression of soils. The settlement can be expressed by the same equation (1) above. NAVFAC(1983) reported the coefficient of secondary compression (C_α) ranged from 0.02 to 0.07. These values are for fills, which have undergone decomposition for about 10-15 years. Higher compressibility is usually associated with high organic content. It is also true for advanced degree of decomposition.

Sowers (1972) introduces a factor "α" for the long-term settlement. He suggested "α" as a function of the initial void ratio (e_o). This "α" value is high if the organic content subject to decay is large and the environment is favorable (i.e. warm and moist, with fluctuating water table that pumps fresh air into the fill). This value is low for more inert materials and under non-favorable environments. Nonetheless, for any given void ratio there is a large range of values for "α" (see figure 4). The relation can be expressed as follow:

For favorable condition to decay	$\alpha = 0.03e_o$
For unfavorable condition to decay	$\alpha = 0.09e_o$

This "α" value can be translated to the classical C_α by dividing "α" by $\{1+e_0\}$ i.e. $C_\alpha = \alpha/\{1+e_0\}$.

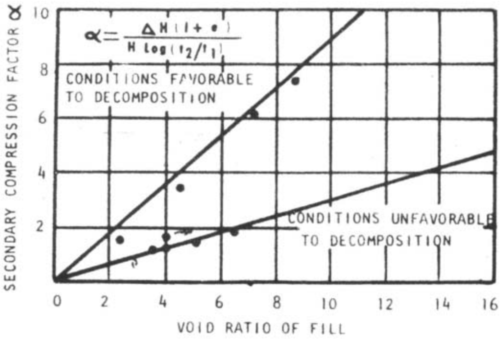


Figure 4

Other calculation methods include the use of a rheological model as presented in the Gibson and Lo theory or the power creep law. The power creep law provides a better representation of the field measured settlement data than the rheological model. However, the rheological model has parameters that can be assigned physical meaning and reflect the effects of certain refuse placement conditions. The details are not presented in this paper.

4.0 DIFFERENTIAL SETTLEMENT & DESIGN MEASURES

There are too many uncertainties for accurate prediction of differential settlement on waste fills. In this case, recourse should be made to the generally accepted rule in engineering practice that, in uniform ground, differential movement will not exceed 75% of the total overall settlements. Thus, once the potential overall settlement has been estimated the likely order of differential movement can be assessed.

Defensive design for buildings demand either the transfer of loading to sound ground through piling or the acceptance of some residual settlement, even after ground improvement, with the load supported directly on the fill. Foundations bearing on fill should be designed to permit settlement without subjecting the superstructure to damaging differential movements or unacceptable tilt.

According to Padfield and Sharrock (1983), most framed buildings can tolerate a differential settlement of about 20mm between columns. This sets the limits for flexible floating supports with individual footings. If this acceptable settlement is likely to be exceeded, a raft foundation for low-rise structures or piling for higher-rise structure shall be considered. The piles are then designed against all adverse features of a refuse waste fills site.

While piling will obviate settlement of the structure, problems may arise from settlement of the fill outside the building area. Service connections and discontinuity of level at the building periphery are particular problems. These problems can be minimized by improving the settlement characteristics of the fill. The treated fill should be sufficiently improved that the loaded areas settle uniformly without imparting significant tilt to the superstructures. Between loaded areas, or between a loaded area and a service run, the differential settlement should be reduced to within a tolerable limits and the service lines should be designed according to the likely settlement profiles.

Dynamic consolidation is a good method of compacting refuse and waste fills. Because void ratio or initial density is related to the initial primary settlement as well as secondary compression, compaction (densification) of fills offers an element of control over potential settlement. However, this method will not eliminate biodegradation and, instead, may provoke or accelerate migration and/or emission of gas (Leach and Goodger, 1991).

5.0 DYNAMIC CONSOLIDATION

The basic concepts of dynamic consolidation (also known as dynamic compaction) as it is used today were presented by Menard and Broise (1975). The method consists of dropping heavy weight ("pounder") weighing 15tons to 20tons from a drop heights of 10m to 20m. using a crawler crane of minimum 100-ton. (Figure 5).

Figure 5

The materials treated by dynamic consolidation exhibit a higher bearing capacity and a lower post-construction settlement. It is most suited for loose coarsed-grained soils (sand and gravel), rubble fills and non-hazardous landfills.

At first sight, the physical performance of dynamic consolidation would appear to be simple i.e. repeatedly dropping a heavy pounder in virtual free fall. However, details such as crane counterweights, jib flexure, torque converter, line pulls, drum size, type and diameter of ropes/cables, clutch, brakes as well as many mechanical properties of the crane have been subjected to rigorous analysis and extensive on-site experiment by the specialist contractors to improve reliability and productivity.

5.1 Compaction Process

The area to be treated is divided into grid patterns with each grid point receiving several blows in a given pass. Several passes may be necessary to obtain the desired results.

It is common to conceptualize the compaction treatment as a series of compaction phases with different combination of energy levels designed to achieve improvement to specific depth. The first phase is generally aimed at compacting the deepest layer by adapting a relatively wide grid pattern and a suitable number of blows. The next phase will involve compacting the middle layer. This is often carried out at the mid-point of the first phase with lesser number of blows and a reduced drop height. The upper surface layer will then be compacted on a continuous overlapping of compaction points from a lower energy blows. It is very often that the upper 0.5 - 1m will not be compacted sufficiently and it is normally roller compacted to finish the compaction works.

5.2 Depth of Influence

Depth of improvement is defined as the depth to which the dynamic consolidation causes some improvement in the fill properties. The pounder weight (W), and the drop height (H), and hence the type of lifting equipment, are selected according to the type of soils to be improved and the depth of improvement required. The relationship between the unit energy and the depth of improvement (D) is given as follow:

$$\mathbf{D = \delta\ \alpha \sqrt{\{W.H\}}\ \ \ldots\ldots\ldots.\ \ (3)}$$

where
D = depth of improvement (m)
W = weight of pounder (ton)
H = drop height (m)
δ = speed factor
(0.9 for cable drop and 1.2 for free fall)
α = soil structure factor
(between 0.3 to 0.7 for different soil types)

The above equation can also be written as:

$$\mathbf{D = n \sqrt{\{W.H\}}\ \ \ldots\ldots\ldots.\ \ (4)}$$

where n = coefficient that accounts for soil type, type of pounder, falling mode etc.

Table 5 lists the proposed values of "n" for different type of soils.

In the case of waste fills, the "n" coefficient can also be estimated from figure 6 (after Van Impe, 1996).

Figure 7 shows the range of compression (enforced settlement) caused by a compaction energy of 625 ton.m. using 25 tons pounder falling from 25m.

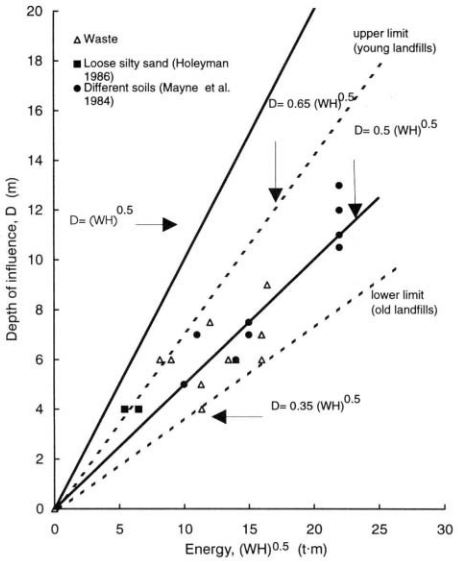


Figure 6

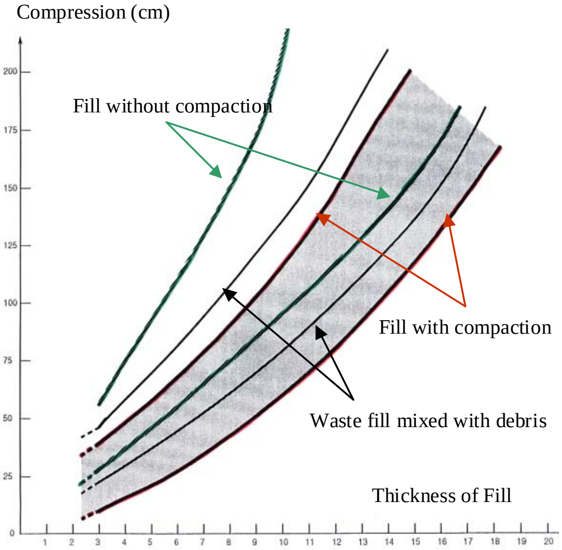


Figure 7

Figure 8 shows the variation of the overall settlement against various compaction energies (W*H). One can see that the age of the landfill is governing this variation.

The effect of different sizes and shapes of the pounder has also an influence on the settlement. A narrow or small cross-section pounder is used specifically to drive fill material down to depth to form columns in soft clays, silts, peats or waste fill (Dynamic Replacement). A larger base area pounder will be more suitable for area compaction.

Table 4

Source	n-values	Soil Type
Menard & Broise (1975)	1.0	all soils
Leonards, Cutter & Holtz (1980)	0.5	-
Smoltczyk (1983)	0.5	soils with unstable structure
	0.67	silts and sands
	1.0	pure frictional soils
Lukas (1980)	0.65 – 0.8	-
Mayne, Jones & Dumas (1984)	0.3 – 0.8	-
Gambin (1985)	0.5 – 1.0	-
Qian (1987)	0.65	fine sand
	0.66	soft clay
	0.55	loess
Van Impe (1989)	0.65	silty sand
	0.35	municipal waste
	0.5	clayey sand
Yee, Setiawan & Baxter (1998)	0.5	calcareous sand / coral sand
Faisal, Yee & Varaksin (1997)	0.33 – 0.39	municipal waste

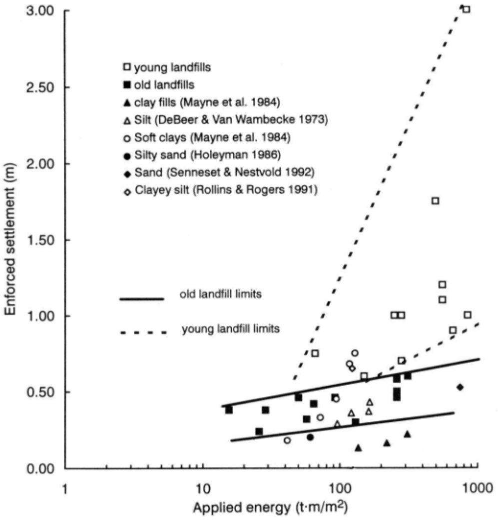


Figure 8

Dynamic replacement (DR) is similar to dynamic consolidation but in this case the compaction process is used to form large diameter granular columns through the soft soils or waste fill to underlying dense bearing stratum (figure 9). This method combines the advantages of dynamic consolidation with those of stone columns. The surface area of the DR column is about 5 to 10m^2 - whereas regular stone columns hardly exceed 0.5 to 1 m^2. The depth however varies between 4 and 8 meters - in case of a pre-excavation dynamic replacement column. The various phases in the construction of granular columns are illustrated in figure 9.

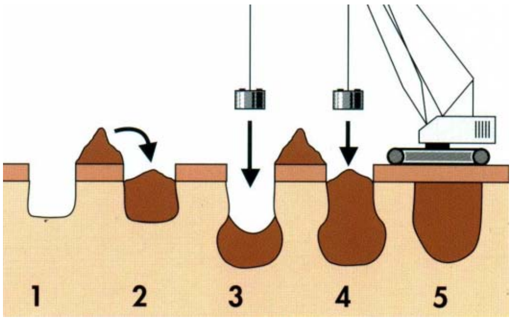


Figure 9

Usually a smaller area pounder is used to facilitate the penetration. Advantage of constructing granular columns by dynamic energy usually results in an increase of about 25% improvement in upper layer of soils between columns and about 50% improvement in the lower layers. Also, high internal shearing resistance results in the granular material within column. These columns also act as oversize vertical drains to reduce the consolidation time as well as to provide better bearing capacity and stability.

6.0 CASE HISTORY: Application of Dynamic Consolidation and Dynamic Replacement for Rehabilitation of a Landfill for a Housing Development Project.

6.1 Project Description and Location

Phase 1 of the project comprised of 108 units of single-storey terrace houses and 102 units of double-storey terrace houses. The site is located at the junction of old limestone and Kenny Hill formation. It was previously a tin mine and was later used as a rubbish dumping ground (figure 10). The overburden therefore consists of poorly compacted household waste whilst the subsurface soil is loose, with slime layers, due to the mining process (figure 11). The site was also scattered with ex-mining ponds.

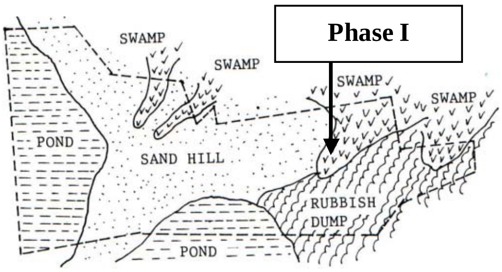


Figure 10

Figure 11(a)

Figure 11(b)

6.2 Subsurface Conditions

Two series of soil investigation were conducted which include deep boreholes, cone penetration tests and pressuremeter tests. The boreholes were sunk to 30m depth terminated upon reaching limestone rock. A sample borelog is given in figure 12. Borehole BH2, which was, located in the middle of the proposed Phase I area indicates a waste fill down to 8.5m. Generally, the thickness of this waste fill (mostly household rubbish) is about 5m to 8m. Underlying the upper waste fill, layers of loose silty clayey sand and clayey silt were found. The water table level was about 1.5 - 2m below the existing ground level. Figure 13 shows the SPT tests results.

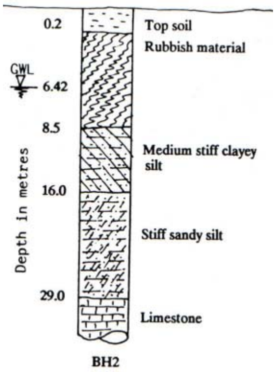


Figure 12

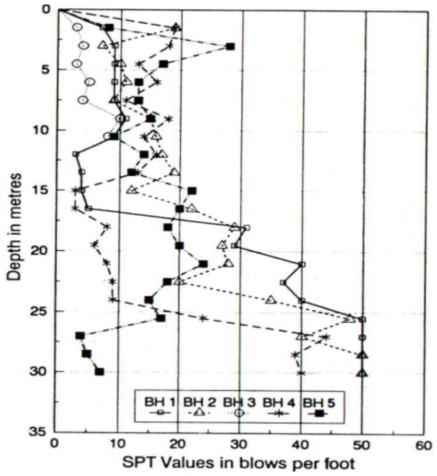


Figure 13

(Continue on page 210)

6.3 Criteria for Ground Improvement

The acceptance criteria for the ground improvement works was:

i) Safe bearing capacity > 120 kN/m²
ii) Differential settlement < 1: 600

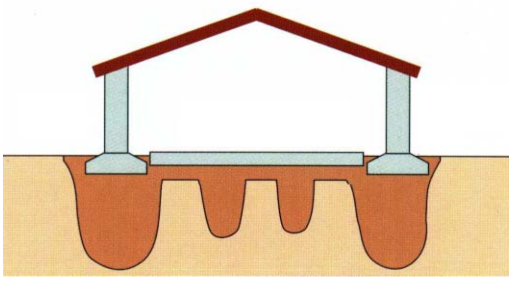
These criteria carried with it a performance guarantee valid for 7 years.

In addition, the structural design also includes structural joints at every 2nd house unit along the terrace row. A shallow foundation design was adopted.

6.4 Design of Treatment, Selection of Plant and Equipment and Construction Sequence

A combined dynamic consolidation and dynamic replacement techniques was used (figure 14). Before the commencement of compaction work, a full-scale calibration test was carried out on site. This calibration test consists of heave and penetration tests. The purpose of the calibration is to:

1) Determine the optimal number of blows for each phase.
2) Determine the required compaction energies and the number of phases of compaction.
3) Check that the pounder penetration is not a volume displacement but a real compaction of the soil.
4) Determine the actual spacing of compaction points to avoid any interference of heaving in between the points.



Dynamic Consolidation to create "compacted sand raft" to allow slab-on-grade and for infrastructure

Dynamic Replacement sand columns to support column loads; increase bearing capacity and reduce settlement

Figure 14

Based of the results of the calibration tests, dynamic replacement and dynamic consolidation were used for the structural area while the dynamic consolidation was used for the infrastructure area (roads and services). The operation parameters were developed and optimized for the compaction operations as shown in Table 5.

A 165-ton crawler crane (American Hoist 9299) was used for the compaction works (figure 15). The effective area of compaction includes a periphery strip (or overwidth) of 5m beyond the boundary of the houses.

Figure 15

The construction sequence was as follow:

Step 1: Excavation of upper 2.5m recent rubbish deposits (refuse < 5 years old).
Step 2: Backfill with 2.5m of clean sand as working platform and also as drainage blanket.
Step 3: Performed dynamic replacement on structural areas (especially below structural columns) and dynamic consolidation over the entire treatment area (Phase 1).
Step 4: Performed phase 2 and ironing phase of ground improvement work.
Step 5: Carry out quality control, instrumentation and monitoring works

during and after ground improvement works.

Step 6: Complete sandfilling to reach finished platform level with compaction to 90% modified proctor standard.

Step 7: Carried out 2m surcharging for 6 weeks. Settlement monitoring.

Step 8: Surcharge removed and proceeds with construction.

6.5 Pressuremeter Test (PMT) and Cone Penetration Test (CPT)

Forty-one locations of PMT test were carried out i.e. 8 locations before compaction works, 12 locations after phase 1, 9 locations after phase 2 and finally 12 locations after the final ironing phase. From the results, it can be shown that there is an increase of the pressure limit P_l and pressuremeter modulus E_m down to about 6-7m depth (figure 16). This increment is comparable with other recorded stiffness modulus found in literature (Table 6).

Sixteen numbers of CPT tests were carried out i.e. 8 nos. before compaction works followed by another 8 nos. after the compaction works. A typical CPT result is given in figure 17. These results confirmed that the compaction effect has reached down to about 6-7m. From the friction ratio, it indicates that the dynamic replacement (DR) columns have managed to penetrate down to about 5m. Below, there was some mixing of sand from the DR columns with the cohesive deposit.

Based on the Menard's equation, the computed "n"-value is about 0.33-0.39 for this particular landfill site.

Table 5: Parameters for compaction

Weight of Pounder = 14 tons (1.83m x 1.83m)			
Maximum Drop Height = 20m			
Spacing of Compaction Points = 6.7m			
Numbers of Phases = 3			
		Infrastructure Area	**Structural Area**
Phase 1	1st pass	8 blows/points (50 tm/m²)	8 blows/points (50 tm/m²)
	2nd pass		6 blows/points (37 tm/m²)
Phase 2	1st pass	8 blows/points (50 tm/m²)	10 blows/points (62 tm/m²)
Phase 3		Ironing phase continuous overlapping prints (87 tm/m²)	ironing phase continuous overlapping prints (87 tm/m²)

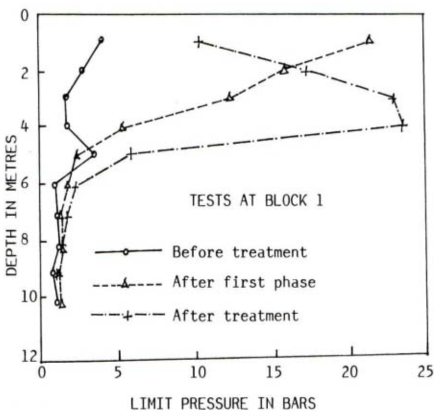


Figure 16(a)

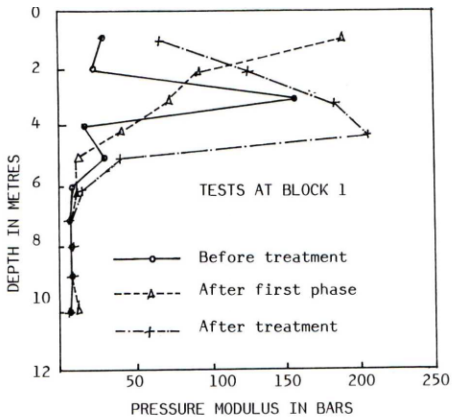


Figure 16(b)

Table 6

Type of Landfill (age)	Average Stiffness Modulus after dynamic consolidation (MPa)	Menard's Rheology factor "α"	Conversion of "in-situ" pressuremeter modulus to stiffness modulus
Domestic waste (< 2 years)	2.5 – 3.0	2.5 – 4.0	$\alpha = E_m / E_y$ where E_y = stiff modulus E_m = pressuremeter moduls α = rheology factor (from load tests)
Domestic waste (> 10 years)	4.0 – 6.0	1.0 – 2.0	
Domestic waste with 30% mineral soil (> 10 years)	5.0 – 8.0	1.2 – 1.5	
Domestic waste with 50% mineral soil (> 10 years)	6.0 – 12.0	1.1 – 1.3	

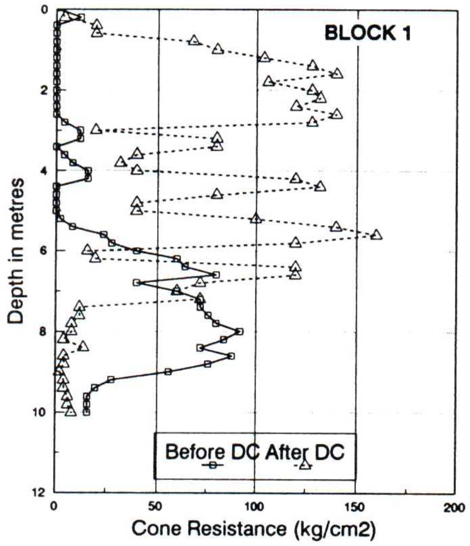


Figure 17

6.6 Enforced Settlement

The enforced settlements obtained were:

Phase 1 : 0.29m
Phase 2 : 0.21m
Ironing phase : 0.10m

The total enforced settlement was about 0.6m which represent about 13-14% of the total thickness of the remaining rubbish deposit after excavation of the upper 2.5-3m.

6.7 Bearing Capacity

The pressuremeter test is a type of load test which in particular yields the limit pressure P_l that corresponds to the failure of the soil. Experience and theory have shown that the ultimate bearing capacity of a foundation is proportional to P_l value. The factor of proportionality so-called the bearing factor K is a function of the relative depth and the foundation shape.

The bearing capacity is calculated according to the D60AN manual for Interpretation and Application of Pressuremeter Test Results to Foundation Design (Sol Soils No: 26 - 1975) - Rule 4 based on equivalent limit pressures.

The equivalent limit pressure P_{le} defined as the geometric mean of the P_l values obtained near to the level of the foundation is given by:

$$P_{le} = \sqrt[3]{\{P_{l1} * P_{l2} * P_{l3}\}} \quad \ldots\ldots\ldots \quad (5)$$

where
P_{l1} is the mean of the limit pressures measured from 0 to 2m depth
P_{l2} is the limit pressures measured from at 3m depth
P_{l3} is the limit pressures measured at 4m depth

The bearing capacity (q) is then calculated using equation (6) below with a bearing factor of K = 0.8 and a factor of safety of 2.5.

$$q = \{P_{le} * 0.8\}/2.5 \quad \ldots\ldots\ldots \quad (6)$$

The calculated safe bearing capacity before compaction works varies from 90 kN/m² to 160 kN/m². After compaction works the calculated safe bearing capacity varies from 320 kN/m² to 500 kN/m² with mean value of 410 kN/m². The bearing capacity is increased by a factor of 3.3.

6.8 Settlement

Estimation of settlement is carried out using the Schmertmann's method based on the cone penetration tests results. The calculated settlement due to a load of 120 kN/m² on a square footing of 1.65 x 1.65m ranges from 8mm to 19mm with a mean value of 12mm after compaction works.

A similar calculation is carried out using the pressuremeter results. The estimated total settlement after compaction works ranges from 5mm to 11mm with a mean value of 8mm.

To obtain the maximum differential settlement between two footings, the worse possible conditions of loading combined with the results of the pressuremeter tests is used. The calculation is based on the following details:

Shape of footing : square
Size of footing : 1.65m x 1.65m
Maximum distance between footing : 2.5m

The computed maximum differential settlement is 1:544.

6.9 Surcharge

Surcharge was carried out after the compaction works to: -

(i) Consolidate the presence of any cohesive layer below the rubbish deposit.
(ii) Reduce the potential differential settlement.
(iii) Reduce future secondary compression.

It was however, primarily used as a simple load test. A surcharge of 2m fill was placed for 6-7 weeks until the time-settlement behavior reached at least 70% degree of consolidation according to field measurements of the settlement plate. The settlement readings taken from 12 sets of settlement plates vary from 4mm to 30mm. Out of the 12 readings, 8 readings have settlement less than 15mm, 3 readings have settlement less than 25mm and only 1 reading has exceeded 25mm. The average value is 13mm.

The 1st phase of the project was completed in 1990. Occupation of the houses was almost immediate and until today (1999) there is no structural defect reported. Figure 18 shows the completed structure after 7 years upon completion.

Figure 18

7.0 CONCLUSION

From the various case histories cited in this paper, the dynamic consolidation technique is applicable for densifying landfill to allow for additional storage space. Furthermore, it is also possible when it combines with dynamic replacement technique to permit developments such as housing projects to be carried out over landfill sites As in any ground improvement projects, instrumentation and monitoring still play a very important role in the success of the works.

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