Soil improvement experiences in Belgium: part I. Overview and dynamic compaction

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A lot of advanced soil improvement techniques can be considered as common practice in Belgium. Two types of procedures have reached a very high technological and engineering design level: soil improvement by vibratory compaction and soil improvement by stone columns. Of the former group of techniques, the experiences with resonant compaction are most promising, and a case history in Antwerp is discussed. Comparisons between resonant vibratory compaction efficiency on the one hand and more traditional vibratory compaction, casing driving compaction and stone column compaction on the other hand are made. This first contribution in a series of three gives an overview of the most commonly used soil improvement techniques including the heavy tamping method, in Belgium. In a second part, practice performance and design aspects concerning two more specific soil improvement techniques (vibrocompaction and stone columns) will be discussed. In a third contribution the overview of soil improvement experiences in Belgium will be completed by some case histories.

Keywords: Anchors & anchorages; case study; drainage; jet grouting; soil nails; stone columns; vibro compaction

Un grand nombre de techniques avancées d'amélioration des sols peuvent être considérées de pratique courante en Belgique. Deux types de procédés ont atteint un niveau très élevé de développement technologique et d'étude technique: l'amélioration des sols par compactage vibrant et l'amélioration des sols par colonnes de pierre. Pour ce qui est du premier groupe de techniques, les plus prometteuses sont celles du compactage à résonance. Les auteurs examineront un cas d'espèce à Anvers et compareront l'efficacité du compactage vibrant à résonance à celle du compactage vibrant classique, du compactage par battage de tubage et du compactage par colonnes de pierre. Le premier des trois exposés donne une vue d'ensemble des techniques d'amélioration des sols le plus couramment utilisées en Belgique, y compris celle du pilonnage. Le deuxième exposé examine les aspects rendement, pratiques et conception de deux techniques d'amélioration des sols en particulier: le compactage vibrant et le compactage par colonnes de pierre. Le troisième exposé donne une vue d'ensemble des expériences d'amélioration des sols en Belgique et présente quelques cas concrets.

General overview

Soil improvement experiences in Belgium

Various ground improvement techniques have been used quite regularly in Belgium over a long period. In Table 1 a general overview of the use of the different ground improvement techniques is given according to the classification of Technical Committee 17 of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE TC-17):

- 'regular' means that the technique is used several times a year;
- 'sporadic' means that the technique is used at least once every 2 or 3 years;
- 'seldom' means that the technique has been used but not with regularity;
- 'never' means that the technique has not yet been used as such in Belgium.

Comments

Dynamic compaction

Dynamic compaction has been used several times in the 1970s and 1980s for the densification of hydraulic fills in harbour areas. The obtained results were normally checked by means of cone penetration tests, and in some cases also by means of pressuremeter tests. From the obtained results it is clear that good densification is obtained for sandy hydraulic fill but not in the underlaying clayey and peaty layers.

Recently, dynamic compaction has also been used for the densification of domestic waste. The obtained densification of the waste materials has been checked by interpretation of

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Table 1. ISSMFE TC-17 soil improvement classification

Classification of TC-17	Regular	Sporadic	Seldom	Never
(1) Ground improvement				
(a) Dynamic compaction			×	
(b) Vibrocompaction	×			
(c) Vacuum consolidation			×	
(d) Drainage	×			
(e) Preloading		×		
(f) Blasting			×	
(g) Heating				×
(h) Freezing		×		
(i) Stone and lime columns	×			
(j) Electro-osmosis			×	
(2) Ground reinforcement				
(a) Reinforced soils		×		
(b) Geosynthetics	×			
(c) Fibre reinforcement				×
(d) Texsol			×	
(e) Mechanically stabilized embankments		×		
(f) Anchorages	×			
(g) Nails	×			
(h) Pinpiles	×			
(i) Diaphragm walls	×			
(3) Ground treatment				
(a) Compaction grouting			×	
(b) Jet grouting	×			
(c) Permation grouting			×	
(d) Hydrofracture grouting				×
(e) Compensation grouting			×	
(f) Fissure grouting		×		
(g) Bulk grouting			×	
(h) Slabjacking	×			
(i) Deep soil mixing			×	
(j) Shallow soil mixing	×			

the wave propagation during compaction according to the spectral analysis of surface waves (SASW) method.

Drainage

Vertical drainage has been in quite common use for more than 30 years for the acceleration of the consolidation, when important fills are located on compressible alluvial layers. Different types of vertical drain have been and are still being used:

- Paper drains (Kjellman type) have been in use for 40 years.
- Sand drains are regularly installed according to the direct-flush drilling method. This type of vertical drain is mostly used for small jobs and in harbour areas.
- Synthetic drains are used very commonly.

Drainage is mostly used for the construction of roads on compressible layers. For the construction of the very-high-speed railway link between France and Brussels, several hundreds of kilometres of vertical drains will be installed. In harbour areas, vertical sand drains are used not only for the acceleration of settlement due to the installation of hydraulic fills above the alluvial deposits but also to prevent a very high groundwater table rise in the upper infilled sand layer.

In recent years, vertical drains have also been installed to accelerate consolidation within new sludge deposits.

Freezing

Ground freezing has been used in Belgium since the beginning of this century, when mine shafts were sunk in the eastern part of the country to depths of 620 m through loose water-bearing formations. For about 30 years, ground freezing has been used quite commonly for civil engineering purposes:

- a full-scale test and several underpinnings have been performed for the construction of the Pre-Metro in Antwerp;
- several ground freezing jobs have taken place in the construction of tunnels and retaining walls in Brussels;
- ground freezing has also been used for the infilling and sealing of old coal mine shafts.

Extensive research programmes have taken place in Belgium concerning the design of ground freezing projects and to evaluate the influence of ground freezing on soil characteristics.

Stone and lime columns

Stone columns are used very commonly in Belgium. Every year, 50–100 km of stone columns are installed by the Keller vibroflotation and the driving methods.

More details about these installation methods and a theoretical approach for their design are given in the second and third articles in this series.

Mechanically stabilized embankments

Piles are used quite commonly for the stabilization of embankments. In most cases, bored piles or screwed piles of the Atlas type are used.

At Geraardsbergen a sliding slope with a natural inclination of 18–22% has been stabilized over a distance of 220 m by means of 150 bored piles with a diameter of 0·81 m. Extensive studies have been performed to calculate the shear forces resisted by the piles, the pressures on the piles, the maximum bending moments in the piles and the necessary length of embedment of the piles. The obtained stabilization of the slope has been checked by means of inclinometers.

Anchorages

Soil or ground anchors are used very commonly for construction pits or other excavations. They are mostly installed to retain pile walls, sheet pile walls, diaphragm walls or underpinning walls. About 90% of the installed anchorages are temporary. For the tendon element both strands and bars are used.

As there are not many unstable slopes in Belgium, only a few instances of slope stabilization with ground anchors exist in this country.

Nails

Soil nailing became very popular in Belgium in 1988 when Smet Boring developed a particular technology for the realization of soil-nailed walls. According to this technology, in the first stage a row of vertical grout columns is installed by the very-high-pressure or jet-grouting technique. The intermediate distance of the grout columns varies between 0·75 and 1·00 m. The diameter of the grout columns is about 0·50 m. The soil nails are also installed by means of the jet-grouting technique. The density of the soil nails normally varies within the range 1 nail per 3–6 m^2.

Very small deformations occur during the installation of the soil-nailed wall. In this way, soil nailing can also be used along existing buildings. In areas where the groundwater level can be lowered without major problems, soil nailing is used for excavations up to a depth of 10 m. During recent years at least 10 000–20 000 m^2 of soil-nailed walls have been installed every year.

In Antwerp an excavation more than 7 m deep has been made around a historic building with an area of 6 × 10 m and a height of 12 m. In the first stage, vertical and subvertical grout columns were installed beside and underneath the existing foundations. The excavation was performed in three stages, and three rows of subhorizontal soil nails have been installed (Fig. 1). During the implementation of this project, levelments were performed regularly. The most important movements occurred during the installation of the first row of nails, when a heave of 4–6 mm was measured. After completion of the excavation, a heave of about 2 mm remained.

Pinpiles

Pinpiles are used very commonly for the underpinning of existing foundations and for the installation of deep foundations in locations with a limited accessibility.

Different types of pinpiles are used:

- small-diameter bored piles installed according to the direct-flush method, with or without additional grouting;
- root piles;
- steel tube piles of a small diameter, installed by ramming, screwing or boring;
- steel tube piles fixed into the soil by cement injection through manchettes.

Diaphragm walls

Diaphragm walls are used very commonly for the construction of retaining walls and for the implementation of very heavy foundation elements. During the 1970s and 1980s, many diaphragm walls were installed during the construction of the Métro tunnels in Brussels and Antwerp. For water-retaining walls, special joints, mostly of the CWS type, are used between the different panels. For almost a decade, diaphragm walls have also been used for the construction of quay walls, even along tidal rivers.

The diaphragm wall technology is also used for the construction of watertight screens in order to limit the influence of groundwater lowerings and for the isolation of contaminated areas. Applying the same technology, antivibration screens can be installed by introducing air mattresses within the excavated trench. Extensive research has been performed by Franki International Technology on this subject.

Jet grouting

For almost 15 years jet grouting has been very popular for underpinning and strengthening foundations and for the construction of retaining walls. In a few cases a watertight screen has been installed by this method.

In Belgium, jet grouting is quite often used for the deepening of existing quay walls, mostly in combination with the installation of ground anchors.

Slabjacking

Slabjacking is frequently used for the repair of concrete pavements for motorways or industrial floors.

SSM (shallow soil mixing)

Shallow soil mixing is applied on a large scale for the construction of embankments for motorways. For the construction of the very-high-speed railway link between France and Brussels, important masses of excavated loam have to be stabilized with lime.

Performance practice and design aspects

Deep dynamic compaction – heavy tamping

The original concept of dynamic consolidation (DC) developed by Ménard has been extended to include dy-

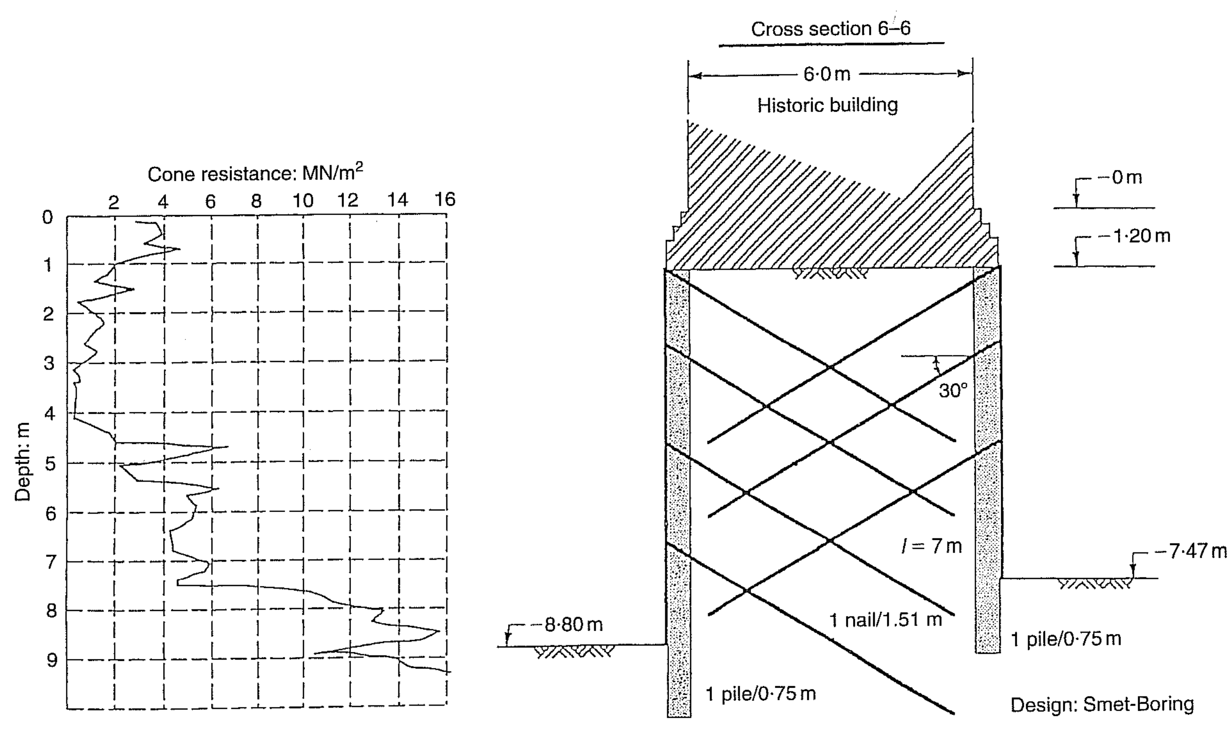


Fig. 1. Soil nailing case history: 'Lombardia' project in Antwerp

namic replacement (DR) and dynamic replacement and mixing (DRM) processes.

The densification of soil following heavy tamping is attributed (Ménard and Broise, 1975) to

(a) compressibility of saturated soil due to the presence of microbubbles;
(b) the gradual transition to liquefaction under repeated impacts;
(c) the rapid dissipation of pore pressures due to high permeability after soil fissuring;
(d) thixotropic recovery.

With successive tampings, energy is imparted to the soil, a certain amount of immediate volumetric strain is mobilized, and excess pore pressure is generated. The level of energy input into the system is called the 'saturation energy' when the pore pressures equal 100% liquefaction pressure. No further volume change can be achieved by imparting additional energy to the soil. Dissipation of pore pressures with time leads to consolidation and gain in strength of the soil. The process of densification under a number of passes with time delays between each pass can be visualized from Fig. 2. The background of the analysis of the heavy tamping mechanism has been described in detail earlier by Van Impe (1992).

For low-velocity impacts on soft cohesive soils the impact energy is used efficiently to improve the soil only in a thin layer. If the impact energy is very high, as in the case of common dynamic consolidation of normally consolidated soils, the depth of influence and the final compacted density are greater, although the energy partly dissipates as radiated longitudinal stress waves.

Beneficial effects such as inhibiting heave and greatly increasing the impact efficiency have been obtained recently in Belgium by the Soils company patented impact block, capable of extending the duration of the pulse on the soil being treated, towards a more 'plastic collision', and allowing implementation of variable block stiffness by prestressing the anchors (Van Impe, 1992).

The extent to which heavy tamping improves the *in situ* soil is one of the primary parameters studied. Ménard and Broise (1975) propose that the depth D up to which improvement is achieved can be estimated from

$$D = n\sqrt{Wh} \quad (1)$$

where W is the weight (in tonnes) and h is the initial height

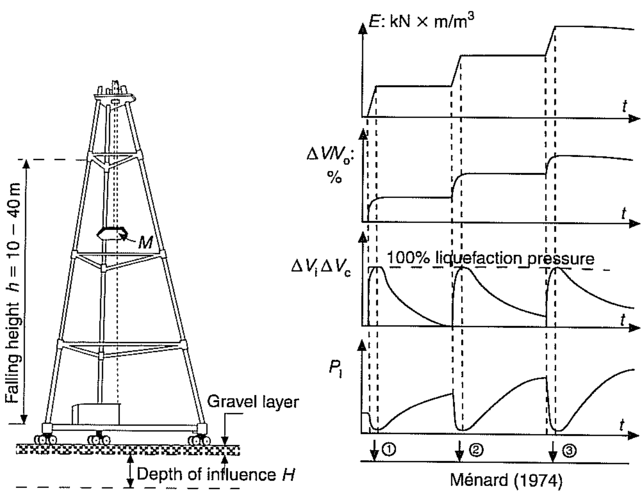


Fig. 2. Heavy tamping principles

(in metres) of the pounder, and n is a constant. Table 2 lists the proposed values of n. Other predictions of the depth of influence, D, of heavy tamping are given in Tables 3 and 4.

However, the depth of influence depends not only upon the energy per blow ($E_d = Wh$) but also on other parameters such as the soil type, site characteristics, surface area and shape of the pounder, number of blows and number of passes, grid spacing, groundwater conditions, etc. An attempt in this direction has been made for cohesive soils by Charles *et al.* (1981), who propose

$$D = 0{\cdot}4\left(\frac{E_d B}{A_p c_u}\right)^{0{\cdot}5} \quad (2)$$

where B is the width or diameter of the pounder, E_d/A_p is the total impact energy applied per unit area of the pounder and c_u is the undrained strength of the soil.

An exhaustive compilation of data from over 120 sites was presented by Mayne *et al.* (1984). Moreover, useful correlations for normalized crater depth, $D_c/\sqrt{Wh}$, overall subsidence of the ground, peak particle velocity, and maximum depth of influence, D, have been presented. The normalized crater depth increases with the number of passes (Fig. 3), the trend showing a limit for this parameter. The overall ground subsidence increases with applied energy (Fig. 4(a)) while peak particle velocity decreases (Fig. 4(b)) with scaled distance, $d/\sqrt{Wh}$. The maximum depth of influence is proportional to the energy per blow (Fig. 4(c)). Within the depth of influence, the maximum improvement takes place between one-third to two-thirds of this zone (Welsh, 1986). If a dense glacial till or hard rock underlies

Table 2. Values of n for deriving the depth of influence D of heavy tamping: $D = n\sqrt{Wh}$

Reference	n
Ménard and Broise (1975)	1·0
Leonards *et al.* (1980)	0·5
Bjolgerud and Han (1963)	1·0 (rockfill)
Smoltczyk (1983)	0·5 (soils with unstable structure)
	0·67 (silts and sands)
	1·0 (purely frictional soils)
Lukas (1980)	0·65–0·8
Mayne *et al.* (1984)	0·3–0·8
Gambin (1984)	0·5–1·0
Qian (1985)	0·65 (fine sand)
	0·66 (soft clay)
	0·55 (loess)
Van Impe (1989)	0·65 (silty sand)
	0·5 (clayey sand)

Table 3. Recommended values for n (Lukas, 1980)

Soil type	Degree of saturation	n
Pervious soil deposits – granular soils	High	0·5
	Low	0·5–0·6
Semi-pervious soil deposits Primarily silts with plasticity index < 8	High	0·35–0·4
	Low	0·4–0·5
Impervious deposits Primarily clayey soils with plasticity index > 8	High	Not recommended
	Low	0·35–0·4 (soils should be at a water content less than the plastic limit)

*Table 4. Proposed values for the depth of influence (adapted from Luongo, 1992)**

Category		Type of soil: Typical applied energy 25 t m/m³ — Pervious cohesionless soil	Typical applied energy 35 t m/m³ — Silty to clayey sands	Typical applied energy > 80 t m/m³ — Partially saturated impervious fill
General	LB			$D = 2{\cdot}0 + 0{\cdot}012\ WH$
	Avg.			$D = 2{\cdot}6 + 0{\cdot}012\ WH$
	UB			$D = 3{\cdot}3 + 0{\cdot}012\ WH$
High water table	LB	$D = 4{\cdot}6 + 0{\cdot}012\ HW$	Insufficient data	Dynamic compaction Not recommended
	Avg.	$D = 7{\cdot}4 + 0{\cdot}012\ HW$		
	UB	$D = 10{\cdot}1 + 0{\cdot}012\ HW$		
Low water table	LB	Insufficient data	$D = 2{\cdot}0 + 0{\cdot}012\ WH$	Same as for category general
	Avg.		$D = 2{\cdot}6 + 0{\cdot}012\ WH$	
	UB		$D = 3{\cdot}3 + 0{\cdot}012\ WH$	
Small tamper base (area 4–5 m²)	LB		$D = -0{\cdot}4 + 0{\cdot}024\ WH$	
	Avg.		$D = 1{\cdot}3 + 0{\cdot}024\ WH$	
	UB		$D = 3{\cdot}0 + 0{\cdot}024\ WH$	

*LB, lower bound; Avg., average; UB, upper bound; H, thickness of layer; W, drop weight (t).

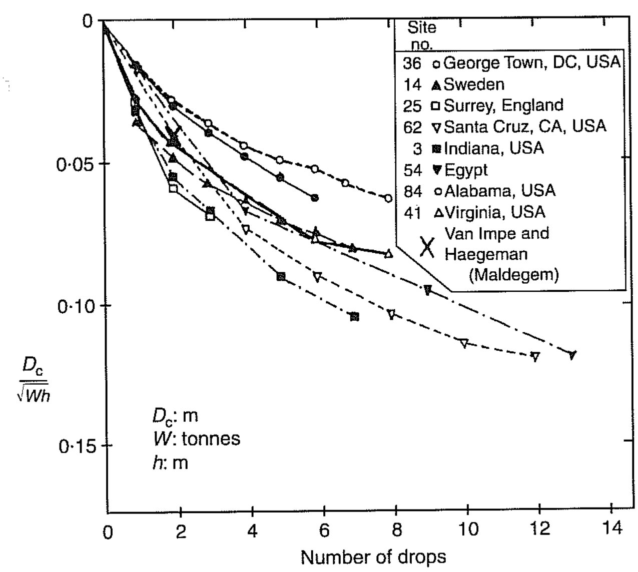


Fig. 3. Normalized crater measurements (after Mayne et al., 1984)

the soft deposit, improvement in the lower half can, however, be more pronounced owing to the reflection of the waves from the bottom. Lukas (1980) points out that multi-tampings at a given location improve the densification in the zone of influence but will not increase the depth of influence. He suggests that the groundwater level should be maintained at least 2 m from the bottom of the dropping weight and, if necessary, partial dewatering of the site may have to be considered. In the case of embankments, one should be also careful in judging the long-term impact improvement.

Heavy tamping, in conclusion, is known to improve the *in situ* properties of the soil. However, it is generally difficult to predict *a priori* the degree and extent of the improvement possible at a given site. At most sites where it is proposed to densify the soil by heavy tamping, initial trials should be carried out in order to estimate the efficiency and degree and extent of improvement.

The weight (W) and height of drop (h) of the pounder, the spacing of impact points (e), the number of passes (n_p) and the number of blows (n_b) per pass are the variables

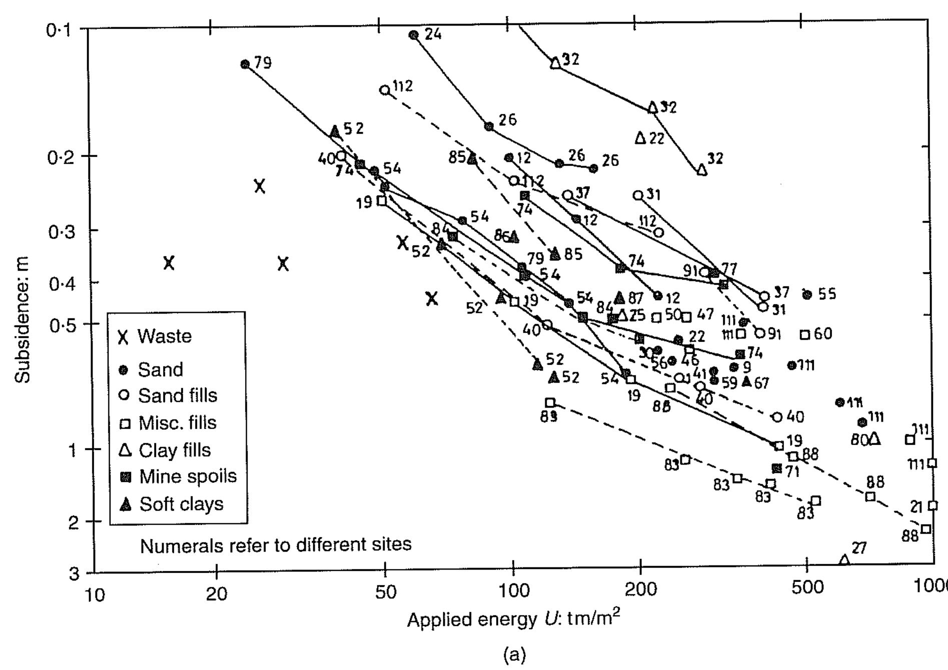
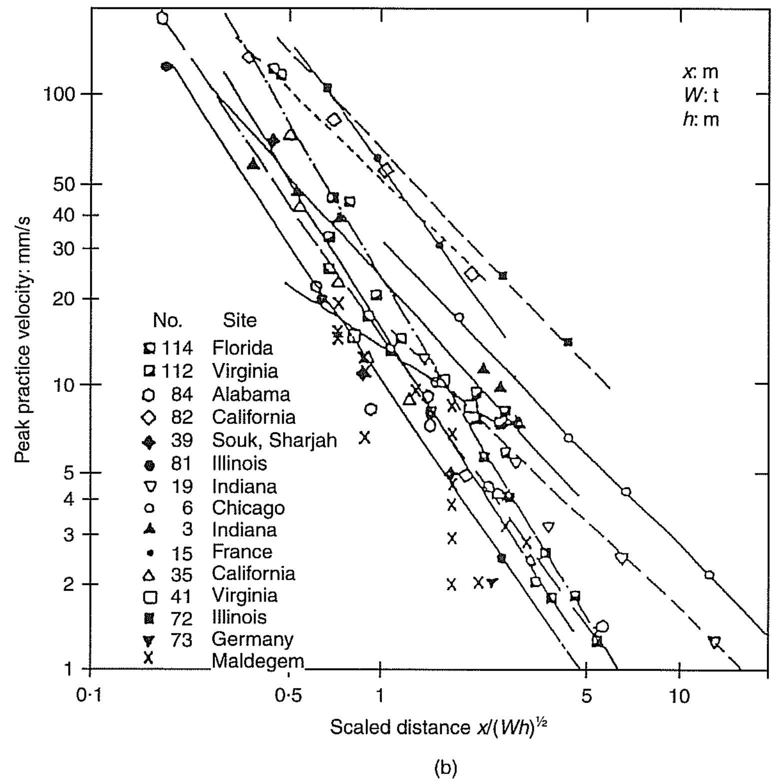
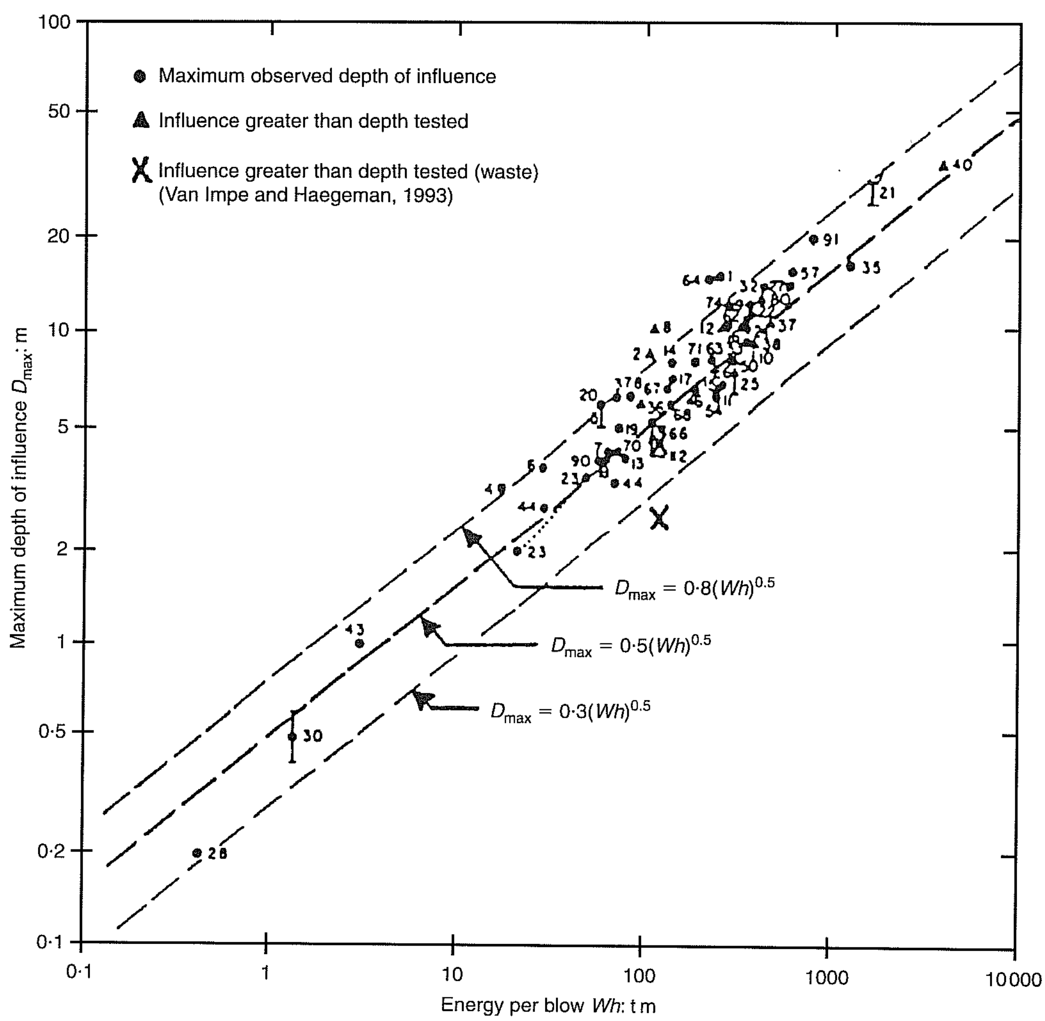


Fig. 4. (a) Observed magnitude of ground subsidence with level of applied energy per unit area (figure continued overleaf)



(b)

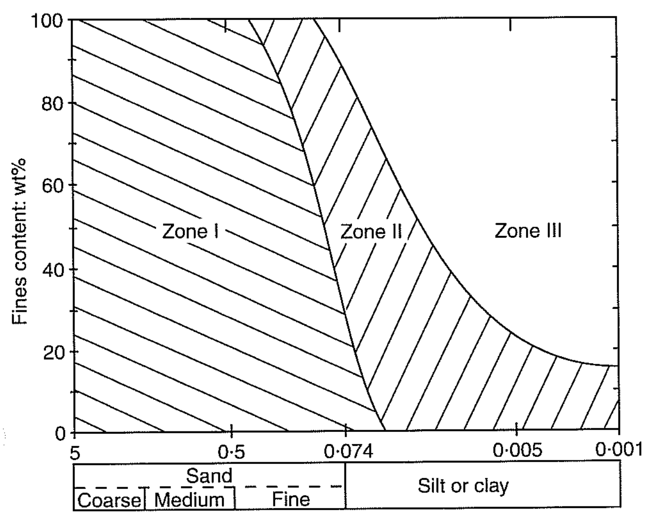


(c)

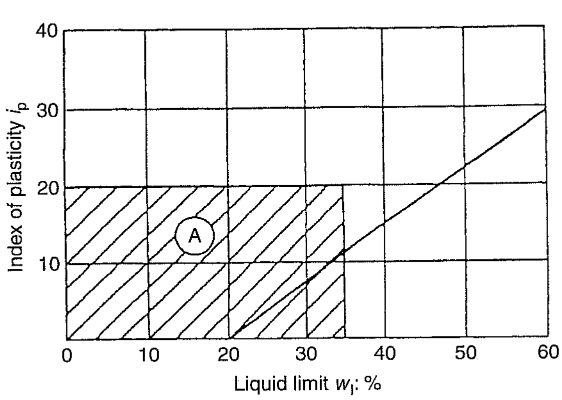
Fig. 4. (b) Attenuation of ground vibrations measured on different dynamic compaction projects; (c) trend between apparent maximum depth of influence and energy per blow (all after Mayne et al., 1984)

available to the engineer at the site. An optimized choice of these parameters should be based on the conditions in the ground before and after a trial treatment. Also, the shape (hydraulic radius) and the relative stiffness of the pounder should play an important role. These factors, so far, have not been properly investigated. Furthermore, the position of the water table and the amount of fines content generally influence the effectiveness of dynamic compaction. Experience to date indicates that the method is less effective when the clay content is greater than 15%, permeability then becoming too low to allow rapid dissipation of excess pore water pressures. In addition, the soil structure may be more difficult to break down owing to intensive water–particle interaction (Mitchell, 1960).

A possible soil grouping for effective use of the heavy tamping method is considered in Fig. 5. In Fig. 5(a) zones I and II are the most favourable ones. Whenever soils in zone III are encountered, it is more appropriate to limit the applicability of heavy tamping (zone A in Fig. 5(b)). As in the case of blasting, heavy tamping necessarily requires additional superficial compaction afterwards, at least over the full crater depth, D_c. The control of the efficiency of heavy tamping, even in a case of municipal waste densification by this method, has recently been done in Belgium using the SASW control of improved soil stiffness method. A detailed description of this case study is given elsewhere (Haegeman and Van Impe 1995).

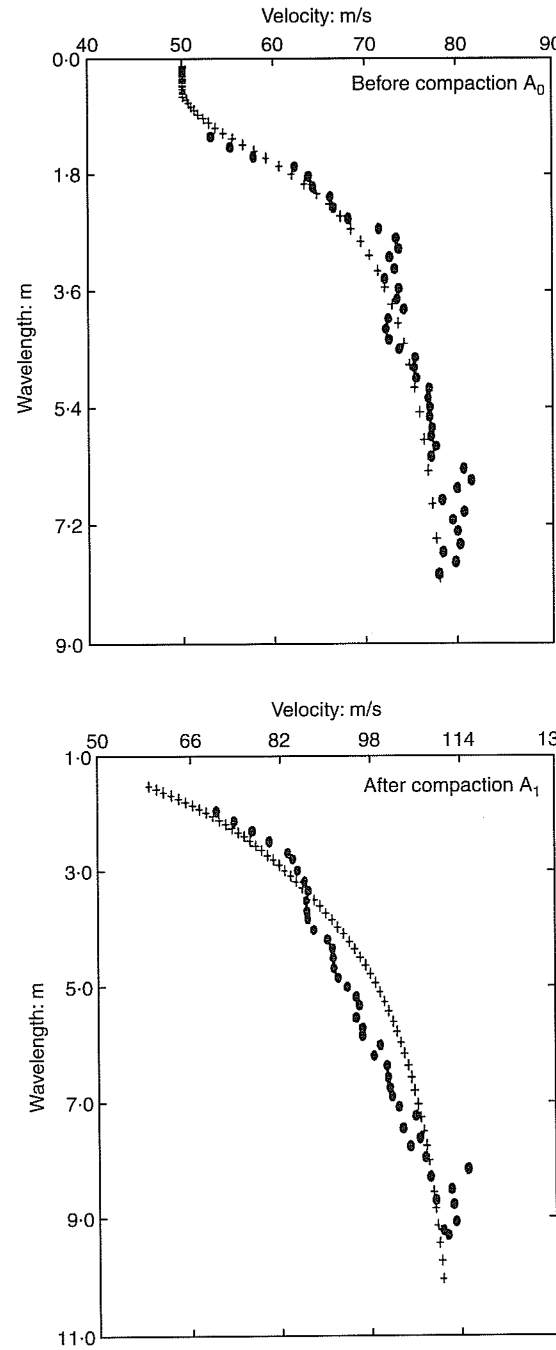


(a)



(b)

Fig. 5. (a) Grouping of soils for heavy tamping; (b) zone of applicability for heavy tamping

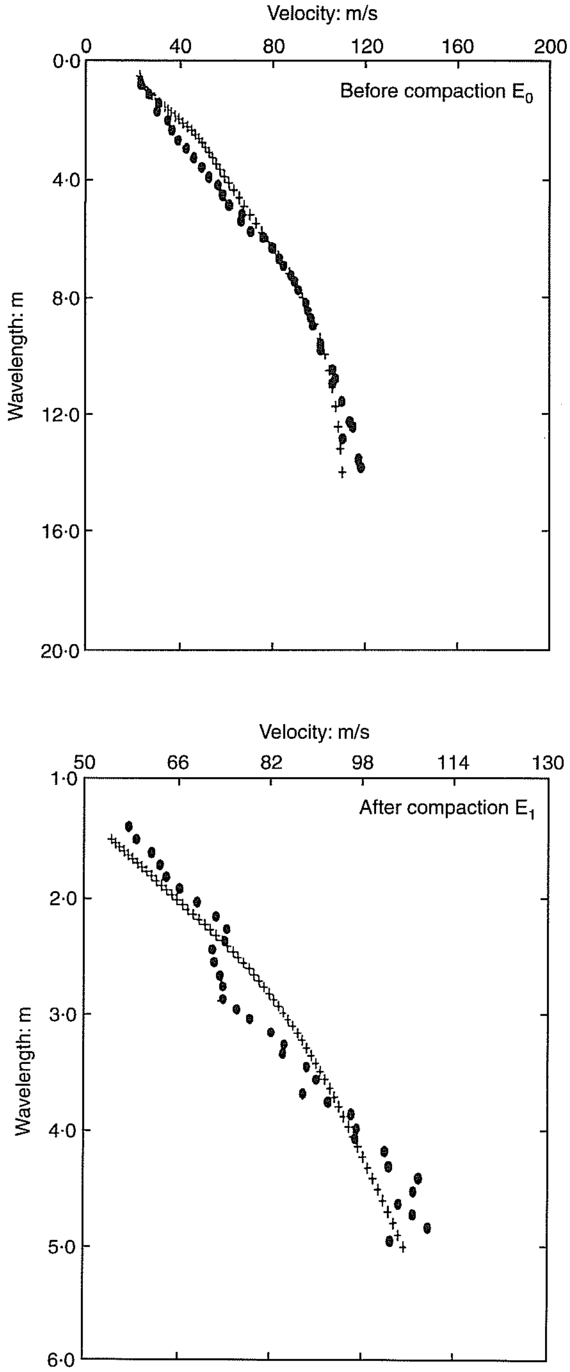


Layer	v_s: m/s		E_y: MPa		E_{PLT}: MPa	
	Before	After	Before	After	Before	After
Upper	55	40	3·63	1·92	1·34	0·71
Deeper	92	130	10·16	20·28	3·76	7·51

(a)

Fig. 6. Rayleigh wave dispersion curves before and after compaction: (a) profile A (figure continued overleaf)

Some field results of these waste disposal SASW tests are the *in situ* Rayleigh wave dispersion curves shown in Figs 6 and 7. In each case, the dispersion curves measured (black dots) before and after dynamic compaction are presented and compared to the theoretically predicted curves (crosses).

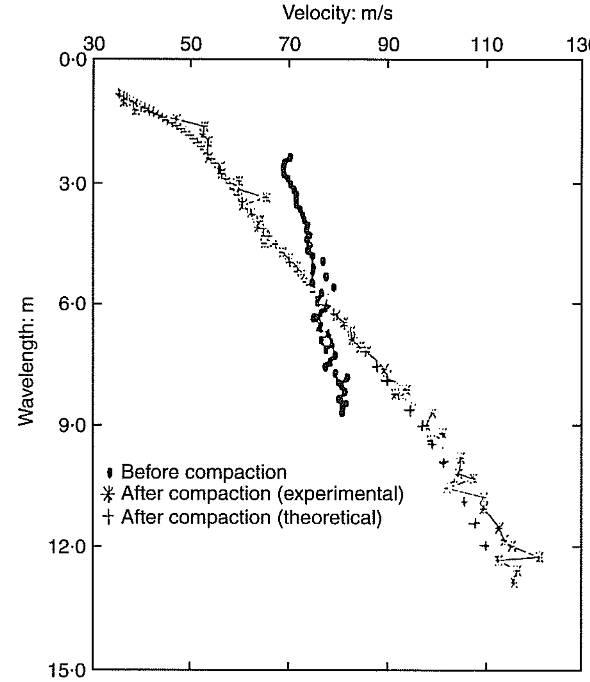


Layer	v_s: m/s		E_y: MPa		E_{PLT}: MPa	
	Before	After	Before	After	Before	After
Upper	25	45	0·75	2·43	0·28	0·90
Deeper	130	160	20·28	30·72	7·51	11·38

(b)

Fig. 6. (b) Profile E (v_s, Poisson's ratio; E_y, Young modulus; E_{PLT}, plate load modulus)

The general trend is that the shear wave velocities after compaction significantly increased at depth with respect to the initial values. The shear modulus increment ranged from about 50 to 100%, and was the highest for the zone with the impact points at a distance of 2 m. Obviously, the top layer, 0·3–0·5 m in thickness, which after a period of heavy rainfall was fully saturated during compaction, was barely compacted. Here, an extra sand layer of 30–40 cm with surface compaction was required in order to improve the surface compaction possibilities. The large differences in some sections of the dispersion curves are indications of the inhomogeneity of the existing waste material.



Layer	Thickness: m	v_s: m/s	E_y: MPa	E_{PLT}: MPa
1	0·4	35	1·47	0·54
2	2	80	7·68	2·84
3	6	190	43·32	16·04

Fig. 7. Profile I before and after compaction

References

Charles J. A., Burford D. and Watts K. S. (1981) Field studies of the effectiveness of dynamic consolidation. *Proc. 10th ICSMFE, Stockholm*, **3**, 617–622.

Haegeman W. and Van Impe W. F. (1995) Evaluation of heavy tamping on a waste disposal by the SASW method. *XI ECSMFE, Copenhagen*.

Lukas R. G. (1980) Densification of loose deposits by pounding. *J. Geotech. Engng. Div. ASCE*, **106**, GT4, 435–446.

Luongo V. (1992) Dynamic compaction: predicting depth of improvement. *Grouting, soil improvement and geosynthetics, geotechnical special publ.*, No. 30. American Society of Civil Engineers, New York, Vol. 2, pp. 927–939.

Mayne P. W., Jones J. S. and Dumas J. C. (1984) Ground response to dynamic compaction. *J. Ground Engng Div. ASCE*, **109**, GE10, 1342–1346.

Ménard L. and Broise Y. (1975) Theoretical and practical aspects of dynamic consolidation. *Geotechnique*, **25**, No. 1, 3–17.

Van Impe W. F. (1989) *Soil improvement techniques and their evolution*. Balkema, Rotterdam.

Van Impe W. F. (1992) Dynamic soil improvement methods. *Proceedings of the seminar on soil dynamics and geotechnical earthquake engineering + workshop on seismic zoning methodologies for geotechnical hazards, 26–29 July, Lisbon*, 84.

Welsh J. P. (1986) *In situ* testing for ground modification techniques. *Use of* in situ *test in geotechnical engineering. Geotechnical special publication*, No. 6. American Society of Civil Engineers, New York, pp. 322–335.

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