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MECHANICAL CHARACTERISTICS OF WASTE DISPOSAL

SUMMARY: The Geotechnics of Waste Landfills is to a large extent governed by the geotechnical properties of waste with particular reference to MSW. One of the major challenges in geotechnical engineering is understanding for the nearly limitless range of soil types that are encountered, how to quantify their important hydraulic properties and stress-strain behaviour in general. If such quantification is not done properly, no soil analysis can provide useful results, and in fact, analysis may generate misleading conclusions.

MEHANSKE KARAKTERISTIKE ODPADNIH MATERIALOV

POVZETEK: Geotehnika deponij odpadnih materialov je v veliki meri vezana na geotehnične lastnosti teh odpadnih materialov s posebnim poudarkom na komunalne odpadke. Eden največjih izzivov v geotehniki za skoraj neštevno mnogo zemljin, ki jih srečujemo, je ravno določanje njihovih hidravličnih lastnosti in odnosov med napetostmi in deformacijami v splošnem. Če ta določitev ni korektno izvedena, nobena geotehnična analiza ne da uporabnih rezultatov, in še več, analize lahko vodijo k napačnim zaključkom. *(prevod urednika)*

INTRODUCTION

The stress-strain behaviour of MSW itself is important in environmental geotechnics because we unavoidably deal in landfill problems not only with soils and rocks, but also with the relevant properties of waste materials to be estimated for the analysis addressing the engineering concerns. However, one has to acknowledge the fact that at present there is little readily available information to assess waste properties, and their variations with time (ex, settlement, stability, etc...).

The quantification of such properties is very difficult because :

1. Municipal solid waste is inherently heterogeneous and variable among different geographic locations,
2. It is difficult to obtain samples of relevant size to be representative of in-situ conditions;
3. There are no generally accepted sampling and testing procedures for waste materials ;
4. The properties of the waste materials change more drastically with time
5. The level of training and education of the personnel on site should be high enough in order to deal with all necessary basic interpretation and understanding of the measurements.

The purpose of this report is to introduce the problems related to physical and mechanical properties of mainly domestic waste

PHYSICAL PROPERTIES OF DOMESTIC SOLID WASTE

Important physical characteristics of domestic waste include moisture content, particle size distribution, organic content and unit weight. However, has to bear in mind the specific characteristics of waste materials. Indeed, waste fill generally consists of many different types of constituents, and these constituents are usually very porous and not fully saturated.
Landva & Clark (1990) pointed out also that since waste fill contains porous constituents, it is necessary to distinguish between intraparticles (within, inside of) and interparticle (between particles) voids. It is entirely possible, for example, to have saturated or partly saturated intraparticle "voids" and dry interparticles "voids" and vice-versa.
In any case the geotechnical classification of waste materials still remains somewhat complex procedure, although it becomes possible to determine reliably the physical characteristics of, for example, the domestic waste.
The determination of index properties then becomes a formidable task. Also, since these materials are often very compressible, properties such as unit weight and permeability must be determined as a function of porosity, which again is a function of the depositing method, of the applied overburden and age of the deposit. Waste types commonly are divided into two groups (ETC8, 1993) :

- soil-like waste, defined as granular waste, for which conventional soil mechanics' testing and soil mechanics theory is to a large extent applicable. In case of MSW however, the applicability of conventional soil mechanics testing and theory are by far more limited.
- other waste, which should be described in such a manner that their mechanical behaviour can be defined, on a case by case basis.

Moisture content in landfills is highly dependent on several interrelated factors including : initial waste composition, local climatic conditions ; landfill operating procedures ; presence of leachate collection systems ; capping history; amount of moisture generated by biological process in the landfill ; and the amount of moisture removed with landfill gases (Mitchell et al., 1995).

Moisture Content

The moisture content of solid wastes (w) can be expressed according to the wet-weight (w_w) or dry-weight (w_d) methods :

$w_w = (W_0 - W_1)/ W_0$ or $w_d = (W_0 - W_1)/W_1$

where W_0 = initial weight of sample as delivered; W_1 = weight of the same sample after drying ; the oven temperature being maintained at 60°C to avoid combustion of volatile material

Huitric et al. (1981) and Tchobanouglous et al. (1993) reported that for most of the domestic landfills under exploitation in the United States, the moisture content varies from 15 % to 40 %, depending on the composition of the waste, the season of the year, and the natural humidity and weather conditions, particularly rain. In regions where evapo-transpiration exceeds precipitation a typical moisture content is on the order of 25 %. Figure 1 shows that in this case history dry moisture content ranges from 22.5 % for fresh waste (non compacted) predominantly of paper and cardboard to around 55 % for 1-5 year old wastes after compression to high densities. It must not be forgotten that intermediate cover layers have a different moisture content, usually far less than that of refuse.

Estimation of the attainable weight density and optimum moisture content through compaction tests is useful in establishing probable values for the specimen's weight density. Figure 2 provides a representative average of the moisture-density relationship conducted in this study using the Standard Proctor Test. The maximum dry weight density was evaluated to be 9.3 kN/m3 at an optimum molding moisture content of 31 %. The variation of the measured weight density with increasing moisture content was similar to that observed for soils. Full saturation was achieved at approximately 70 % water content with a weight density of approximately 8 kN/m^3 . A weight density of 12 kN/m^3 was estimated from the zero air void curve at a moisture content of 31 %.

Unit Weight

Unit weight will vary throughout any landfill and is difficult to determine because of variabilities in composition, method of placement, induced ageing, depth , and local moisture content. Numerous values are given throughout the literature (fig.3a). Fassett et al. (1994) reported for example unit weights that range from 3 to 9 kN/m^3 for fills that have received poor compaction, 5 to 8 kN/m^3 for moderate compaction, and 9 to 10.5 kN/m^3 for fills with good compaction. Van Impe (1993, 1994) reported values ranging from 5 kN/m^3 to 10 kN/m^3 for some Belgian landfills (table n° 1), which is fitting into the range of other available data, fig. 3b.

Table 1 : Unit weight values from some Belgian Landfills (Van Impe,1993, 1994).

Landfill	Comments	Unit-Weight (kN/m^3)
Dendermonde	Freshly deposited paper waste	5
	compacted milled paper waste	8
	compressed blocs of milled paper waste	8
Maldegem	densified municipal waste (heavy tamping)	10

One alternative to compute the unit weight is proposed by Landva & Clark (1990) and takes into account the intraparticle and interparticle voids. An instructive example given by Landva & Clark (1990) is an aggregate of metal cans. If made of sheet steel, the unit weight of the solid (or the specific weight of the steel material) would be about 80 kN/m^3, while the unit weight of the empty can without inter can porosity could be as low as 2 kN/m^3 (for a porosity of 97.5 %) and, in the same condition that of a water-filled can would be about 12 kN/m^3. With an interparticle (i.e. inter-can) porosity of around 40 %, the unit weights of the aggregate would be approximately varying from 1 kN/m^3 to 11 kN/m^3.

The average unit weight of the individual constituents (i) of the waste depends on the unit weight of the solid portion of each constituent and on its porosity and degree of saturation. In general the average unit weight of the (n) constituents is :

$$\gamma_c = \frac{1}{\sum_1^n \frac{w_i}{w_c} \cdot \frac{1}{\gamma_i}}$$

where : γ_i = unit weight of constituent u, and w_i/w_c = weight of constituent i as a fraction of the total weight w_c of the constituents,

On exposure to water, the unit weight of any constituent absorbing water would increase (e.g. that of food waste, garden refuse, paper, textiles, wood, etc). The new average unit weight of the constituents could be expressed as :

$$\gamma'_c = \gamma_c \left[1 + \sum_1^n \frac{w_i}{w_c} \cdot \frac{\Delta\gamma_i}{\gamma_i} \right]$$

where $\Delta\gamma_i$ = increase in unit weight of constituent i.

Using the above method, suggested unit weight values ranging from 7 to 14 kN/m^3 depending on composition and water content can be proposed. It is generally believed that unit weight increases with depth. Indeed, the initial unit weight will increase with compression immediately following application of overburden pressure due to waste placement. The unit weight may also increase with the additional compression that occurs over time.

Particle Size Distribution

As discussed earlier, wastes can be divided into two groups :

1. Soil-like,
2. other wastes

In case 1 soil mechanics procedures for laboratory tests and calculation are commonly used. For the case of solid municipal waste, it is difficult to use soil mechanics methods.

One first approach is to try to identify the waste by running a gradation curve (Jessberger, 1994) for the various portions depending on the so called equivalent sieve opening size, ending up with gradation curves somewhat similar to soil. Fig. 4 also show the results of grain size analysis on different types of municipal wastes. It is interesting to note the tendency of the fine grained material amount to increase with ageing of waste. This tendency can be explained by the different states of decomposition that waste is undergoing. Tchobanouglous et al. (1993) proposed a similar method based on the ability of a waste component to pass through a sieve. Typical data on the size distribution of the individual components in domestic waste are given in fig. 5. Both approaches can help, as a first step, the identification of the waste from a soil mechanics point of view.

Permeability of Waste

New solid waste facilities are required to install liner and in-situ leachate control and collection systems. In this case proper assessment of the hydraulic characteristics of the waste itself is an important design element because of the potential impacts related to uncontrolled migration of leachate and the stability problem. Table 2 provides published determinations of the hydraulic conductivity for wastes; at first sight it seems that the measured values could be associated with clean fine sand. One has also to keep in mind that these values are influenced by the degree of compaction, waste ageing etc... One therefore needs to assess the hydraulic conductivity on a case-by-case basis. However, it seems that a value of 10^{-5} m/s can be suggested as a first approximation.

Table 2 : Summary of determination of hydraulic conductivity of domestic waste

Source	Unit weight (kN/m^3)	Hydraulic conductivity (m/s)	Method
Fungaroli et al. (1979)*	1.1-4 (milled waste)	10^{-5} to $2x10^{-4}$	Lysimetres determination
Koriates et al. (1983)*	8.6	$5.1x10^{-5}$to $3.15x10^{-5}$	Laboratory tests
Oweis & Khera (1986)*	6.45	10^{-5}	Estimation from field data
Oweis et al. (1990)*	6.45 9.4-14 6.3-9.4	10^{-5} $1.5x10^{-6}$ $1.1x10^{-5}$	Pumping test Falling head field tests Test pit
Landva & Clark (1990)*	10.1-14.4	$1x10^{-5}$to $4x10^{-4}$	Test pit
Gabr & Valero (1995)	-	10^{-7} to 10^{-5}	Laboratory tests
Blengino et al (1996)	9-11	3.10^{-7} to 3.10^{-6}	Deep boreholes (30-40m). Falling head field tests
Manassero (1990)	8-10	$1.5.10^{-5}$ to $2.6.10^{-4}$	Pumping tests (15 + 20 in depth)
Beaven & Powrie (1995)	5-13	10^{-7} to 10^{-4}	Laboratory tests under contining pressure from 0 to 600 kPa
Brandl (1990)	11-14 (roller comp.) 13-16 (roller+DC)	$2.10^{-5}$to 7.10^{-6} 5.10^{-6} to 3.10^{-7}	Falling head field tests Test pit
Brandl (1994)	9-12 (pretreated)	2.10^{-5} to 1.10^{-6}	Laboratory tests
Brandl (1994)	9-12 (pretreated)	5.10^{-4} to 3.10^{-5}	Laboratory tests
Brandl (1994)	13-17 (very compacted)	2.10^{-6} to 3.10^{-8}	Laboratory tests

* = from Oweiss et als. (1990)

MECHANICAL PROPERTIES OF DOMESTIC SOLID WASTE

Most of the literature data on waste mechanical and deformation characteristics result from "estimation", few data are derived from back-analysis or from measurements out of laboratory or in situ tests.

The mechanical properties like shear strength and compressibility are dependent on the individual composition of the waste material and on the mechanical properties of its constituents. In addition the mechanical parameters are time-dependent and related to the state of decomposition. In order to provide applicable parameters for stability or deformation analysis it might be useful to conduct appropriate tests consistent with the cinematics of potential failure problem, for the specific cases ; anyhow, all geotechnical parameters have in any case to be implemented with engineered judgement.

The interpretation of the results of the tests on waste remains subjected to many uncertainties, due to the lack of a conceptual reference model of behaviour for this material. The analysis is usually still made starting from models and methods established for soils possibly with some reinforcement. In fact, most wastes are composed of individual "particles" with a certain interlocking sheets and for strips in plastics or textile ; and this supports this approach to some extent. However, there are significant differences with soils ; the void ratio is very high, which implies an unusually large volumetric compressibility ; the "particles are of very different natures, and some of them are weak and very deformable or crushable; there is a process of decomposition with time, which causes unusual auto-consolidation and a variation of properties with time.

Deformation characteristics and behaviour

The mechanisms governing domestic waste settlement are many and complex, even more so than for a soil due to the extreme heterogeneity of the waste, their own "particle" deformability, and the large voids present in the initial refuse fill. The main mechanisms involved in refuse settlement are discussed by some authors, Sowers, (1973), Huitric (1981) and Gilbert & Murphy (1987), Van Impe & Bouazza (1996).

Our general proposal can be summarized as :

1. Physical compression due to mechanical distortion, bending, crushing and reorientation.
2. Ravelling settlement due to migration of small particles into voids among large particles.
3. Viscous behaviour and consolidation phenomena involving both solid skeleton and single particles or components,
4. Decomposition settlement due to the biodegradation of the organic components
5. Collapse of components due to physico-chemical changes such as corrosion oxidation ; degradation of inorganic components.

This proposal for subdividing the MSW load settlement curve also closely matches the indications in fig. 6 (adapted from a proposal by Grisolia et al. 1992).

The factors affecting the magnitude of settlement (under own weight as well as under overloads) are many and are influenced by each other (Edil et al., 1990). They include :

1. initial domestic waste density or void ratio ;
2. content of the decomposable materials in the domestic waste ;
3. fill height ;
4. stress history ; (treatment during and after emplacement)
5. leachate level and fluctuation ;
6. environmental factors (such as moisture content, temperature and gases present or generated within the landfill).

The term consolidation in the above suggested steps of MSW load-settlement curves, refers to settlement resulting from the dewatering of the freshly deposited saturated materials.

Generally, the final settlement of refuse fill is characteristically irregular. Initially, there is a settlement within one or two months after completing construction, followed by a substantial amount of secondary compression over an extended period of time. The rate of settlement decreases over time and with increasing depth below the surface of the fill. Under its own weight, refuse settlement typically ranges from 5% to 30% of the original thickness with most of the settlement occurring in the first year or two. This is also confirmed by tests results suggested by Gandolla et al (1995), fig. 7.

The consolidation, compressibility of the solid waste skeleton is commonly estimated using the theory of one-dimensional consolidation, with the total settlement divided into primary

and secondary components. Properties necessary for settlement analysis include the compression index (C_c), or compression ratio CR, (C'_c) to estimate primary settlement, and the secondary compression index (C_α) or the modified secondary compression index ($C_{\alpha\varepsilon}$), or C'_α used to estimate the settlement that occurs after completion of primary settlement while the waste is subjected to a constant load. Unlike soils, the compaction or secondary compression of municipal landfill waste includes contributions from biological or chemical "solid" decomposition, as well as from creep.

For the analysis of immediate load induced compressibility (consolidation), methods derived from Soil Mechanics are followed either using elastic approaches or e-log σ' laws.

For the analysis of the compaction or delayed settlements, there are still few direct evidences of their final values, due to the lack of long enough observation periods and/or to uncertainties about the early stages of old landfills (precise dates dumping rate and elevations, etc..). For a self weight action only, several authors quote final "compaction" settlements in the range 10-40 % of the landfill thickness, depending on the type of waste and degree of compaction achieved at the time of placement (Edil et al., 1990, Frantzis, 1991, Leach & Goodger, 1991).

As rightly pointed out by Sagaseta (1993) theoretical analysis of the "compaction" or delayed settlement is difficult, due to the interaction of very geotechnically unusual mechanisms (creep, internal biodegradation) which follow various laws and are governed by very different factors. Hence, only phenomenological approaches have been followed as far. A number of time-settlement laws have been proposed to match actual observations.

Some desirable features of these fitting laws can be stated as follows :

- to be dimensionally correct
- to be defined by a short number of parameters (1-3 is a usual range)
- these parameters having a physical meaning, or being related to some known properties, so that reasonable ranges can be given to them
- to give reasonable predictions for long time intervals
- to separate the influence of as many relevant factors

In deposits where internal pore pressures can freely dissipate, such as in most municipal solid waste landfills, the bulk of primary settlement may occur so quickly as to be concurrent with the construction operations increasing the load. Therefore, during initial waste placement, primary settlements caused by self weight of the waste will occur as the load is applied and are believed to be substantially complete upon cessation of waste placement activities.

We believe that the codes for modeling waste settlement behaviour are still far from an acceptable level; much more fundamental research will be needed before this kind of problem can be handled even modestly. More on such modeling is added further.

Model Predictions for settlements

A soil mechanics modeling has still been generally adopted in landfill engineering practice to predict the projected settlements of a landfill subjected to surface loading.

Sowers (1973) adopted a behaviour similar to secondary consolidation of soils, in which the settlements were assumed to be proportional to the landfill thickness, H, and to vary linearly with log of time. The secondary compression was given as :

$$S_s = C_\alpha \, H. \log \frac{t_1}{t_2}$$

t_1 & t_2 being the time interval.

More recently two similar studies were reported by Bjarngard & Edgers (1990) and Fassett et al (1994), in which the respective researchers compiled available reported MSW landfill performance data, evaluated the data, and proposed empirical models for the prediction of settlements in landfills.

The model proposed by Bjarngard & Edgers (1990) is presented in fig. 8 or :

$$\frac{\Delta H}{H} = CR\log\frac{\overline{P_0}+\Delta p}{\overline{P_0}} + C_{\alpha\varepsilon_{(1)}}\log\frac{t_{(2)}}{t_{(1)}} + C_{\alpha\varepsilon_{(2)}}\log\frac{t_{(3)}}{t_{(2)}}$$

where ΔH= settlement ; H= initial thickness of waste layer ; $\Delta H/H$= vertical strain (or normalised settlement) ; $\overline{P_0}$ = initial average vertical effective stress at the considered depth; Δp= average induced vertical effective stress also due to overburden increase at considered depth ; $t_{(1)}$= time (days) for completion of "initial" compression as defined in

fig. 8 ; $t_{(2)}$= time (days) for completion of "intermediately" compression as defined in fig. 8; $t_{(3)}$= period of time (days) for prediction of settlement as defined in fig. 8 ; CR= (compression ratio) = $C'_c = \Delta\varepsilon/\Delta\log\sigma$; $C_{\alpha\varepsilon(1)}$= intermediate secondary compression index; $C_{\alpha\varepsilon(2)}$= secondary compression index.

Comments on deformation behaviour

It may be noted that the settlement equations are expressed in terms of effective stress as an extension of "soil mechanics" principles. In a typical landfill, the waste mass is generally in a moist state, except for a limited zone at the bottom which may be saturated with leachate. Accordingly, it is mostly also realistic to go out from total stresses.

Settlement predictions for solid waste landfills are complicated due to the random nature and decompositional characteristics of the waste, short-term and long-term environmental conditions, and operational methods. Additionally unpredictable differential settlement can occur over relatively short distances from deterioration or collapse of containers, appliances, and similar materials. Settlement predictions can be further complicated for landfills constructed on compressible foundations that may exhibit complex settlement characteristics. Although landfill settlement is frequently evaluated using the theory of one-dimensional consolidation, this approach is complicated since :

- the use of compression and recompression ratios are dependent on initial values of e or H and these properties are often not reliably known
- the (e) vs. log(p) or log (t) relationship are often not linear and therefore, compression coefficients C_c and C_α vary considerably with the initial stresses within the landfill while these stresses change with time (Fassett et al. 1994) ;
- the amount of primary settlement depends on the effective stress, which in turn is a function of refuse unit weight and the level of leachate within the landfill (both of which may be poorly known and may change with time).

Many inaccuracies are involved in the evaluation of time dependent compressibility of municipal solid waste and the validity of results calculated using a classical consolidation approach are questionable at best. In the design stage, however, conservative values for the different compressibility parameters could be used to provide an estimate of the general magnitude of deformation that might be expected during the various phases and active life and post-closure period of the site.

Perhaps of greater concern to the designer than total settlement would be the impacts of differential settlements on the landfill liner, cover, and environmental control systems, as these systems are composed of materials with markedly different properties and behaviour (Jessberger 1994a). As a practical matter, in many instances, differential settlement could be conservatively estimated as a percentage of waste thickness at various points in the fill and the tolerance of the different environmental control system to these settlements could then be evaluated.

A one-dimensional consolidation process can be simulated in the laboratory by compressing a waste specimen in a special testing apparatus, on condition one can choose the appropriate sample dimensions. Published records of laboratory measurements of waste settlement are very scarce. However, some were reported recently. Landva & Clark (1990) compressed old wastes (age unknown) from different sites in a 0.5 m diameter consolidometer. The results are given in fig. 9 and show the high compressibility of the waste. Values of CR in the range of 0.2 to 0.5, depending on the stress level, were reported. Whereas $C_{\alpha\varepsilon}$ was found to be in the range of 0.2 % to 3 % per log cycle of time; and it appeared also that $C_{\alpha\varepsilon}$ increased with increasing organic content.

A proposal for the $C_{\alpha\varepsilon}$ values estimation in case of MSW, is given in fig. 10.

Compression behaviour of mixed or municipal waste can be assessed by the use of various field tests, each with their own specific application boundaries, such as plate load tests for surface layers, pressuremeter tests in depth, more advanced spectral analysis of surface waves (Van Impe et al 1993) and overall direct settlement measurements.

The settlements could be measured at different landfill levels so that the compression of the interlayered strata would be analysed separately. This would already be an important improvement to the general settlement follow-up.

Plate loading tests are usually performed on top of landfills after closure, once the final sealing cover has been installed. The usual aim is to evaluate the bearing capacity for potential "constructional" use of the landfill area. In some case, tests have been run during landfill operations, in order to analyse the mechanical behaviour of the waste material.

To be meaningful, some tests have to be located on the weakest and some on the strongest areas of the fill. Typical results of plate ($\varnothing$= 0.6 m) load test conducted at the Appels landfill, Dendermonde (Belgium) are shown in fig. 11. There is no failure evidence on this fresh MSW. The load settlement curves remained straight until the maximum applied load was reached.

At this stage, a geotechnical parameter describing the compressive behaviour of municipal waste, (the stiffness modulus), still is not enough used. It can be determined either by the use of laboratory tests, back analysis of landfill settlements or better from relevant field testing at corresponding strain levels. Nowadays, the small strain overall stiffness moduli for municipal and other waste disposal sites, in many cases can reliably be estimated with non-destructive methods such as Spectral Analysis of Surface Waves (SASW). Fig. 12 shows the load dependent range (at various levels of strain) for the stiffness modulus of MSW reported by several authors. The variation of the stiffness modulus is delimited by an upper bound and a lower bound depending on several factors such as waste composition, state of compaction, soil cover, plate diameter (in some cases), type of tests and ageing. It should be pointed out that, according to the experiences recorded by Haegeman & Van Impe (1995) and Haegeman (1995) the SASW values can be assumed to show some 30 % decay when translated in plate load test results.

A first estimation of the (low organic content MSW) secondary compression rate by the ratio C_α/C_c = 0.075-0.17 is very tempting. However, such a relationship must be used with great caution because of the additional and complex effect of time and biodegradation of waste. The ratio C_α /C_c in case of severe biodegradation can rise up to 1.

Furthermore, after the landfill has been closed and sealed, its bearing capacity as foundation will depend on the remaining long term settlement rate. For these purposes, the subsidence measurements with different techniques such as aerial survey, optical survey, photogrammetry could be used.

Static Shear Resistance

Shear resistance is a geotechnical parameter of primary concern in describing the properties of domestic solid wastes. Equivalent to soil mechanics, shear angle or angle of internal friction, ϕ, and

"intercept cohesion", c, are used in design calculation.

Four general approaches are used to estimate the shear strength of domestic solid wastes;

1. laboratory testing in TX-conditions
2. back calculation from field tests and operational records
3. in-situ testing ;
4. direct shear tests of large dimensions

Laboratory testing has been performed on reconstituted samples from landfills, samples in which various substitution were made, and on samples collected from drive samplers. Large triaxial compression cells or large direct shear apparatus can be used. As specified earlier, interpretation of the tests on waste by means of concepts derived from soil behaviour can be useful, at least at the present state of knowledge. On this basis, the concepts of shear angle, and "cohesion" intercept are commonly used. On the other hand, wastes are usually non saturated. Therefore, interpretation of test results in terms of undrained situation with no volume change (ϕ = 0) may be a very unrealistic approach and an analysis in terms of equivalent c "intercept" - ϕ can be more adequate (Sanchez-Alciturri et al. 1993a).

From typical triaxial results shown in fig. 13 one can observe that the stress-strain relationship shows no peak data unlike tests with soil materials. As matter of fact domestic waste material can sustain very large shear strains without reaching failure. Moreover, at large strains a slight upward inflection is produced suggesting that the material is stiffening.

At early stages, the horizontal strain fig. 14 is about zero. At the same time a high vertical compression takes place (progressive collapse of the waste). This might result in an increase of the effective contact surface ; implementing that the stresses acting on these surfaces probably do not change significantly.

On the basis of TX results on MSW, it is apparent that the ultimate state boundary surface at failure cannot be clearly defined because of reachable strain limits of this kind of test equipments. Nevertheless,.it is obvious that "failure" parameters ϕ and c according to the

Mohr-Coulomb criteria are for operational problems, (stability analysis...) more reliably defined on the basis of allowable strain (p-q-e-envelopes varying with shear strain level).

It is on the other hand also quite obvious that moisture content is playing a predominant role (fig. 15).

In general, it can be assumed that most of the MSW shows a homogeneous composition over a certain volume of the waste body (minimum representative volume) and in most cases reinforced elements are randomly distributed over this area so that the waste can be regarded as isotropic material. Kockel & Jessberger (1995) showed that the shear strength of the basic matrix is primarily of the friction type with a maximum φ value of 42° to 45° only activated at very large strain and slightly influenced by the reinforcing plastics (fig. 16). The intercept cohesion is particularly depending on the reinforcing matrix and may be defined as a "cohesion due to tensile strength" of the reinforcing components (fig. 17). The activation requires, however, large strains and starts at about ε = 20 % when φ is almost fully mobilized.

Direct shear tests

A direct shear test on waste is conducted in order to obtain the shear resistance parameters of the waste (table 3); or in some cases the surface contact between compacted bales of refuse (Van Impe, 1993, Del Greco & Oggeri, 1994). This type of test does not reproduce the real behaviour of waste in a landfill but it provides an initial approach to more careful procedures.

Table 3 : Kavazanjian et al., 1995

Reference	Data Type	Results	Comments
Landva and Clark(1990)	Laboratory direct shear tests on MSW	φ=24°,c=22kPa to φ=39°,c=19 kPa	Normal stresses up to 480 kPa.
Richardson and Reynolds (1991)	Large direct shear tests performed in site	φ=18°to43° and c=10kPa	Normal stresses range from 14 to 38 kPa. Unit weight of waste and cover estimated as 15 kN/m^3

Typical stress-strain curves obtained from this type of tests are shown in fig. 18. Both specimen exhibited continued strength gain at well excess of 10 % strain, none of the specimen tests reached peak strength. In this case, similarly to the triaxial test, the allowable strain concept is applied. The values of c and ϕ deduced from the measured data are usually evaluated at 10 % and 15 % strain. Another interesting feature can also be observed in fig. 18. Indeed, the discrepancy in the results is quite striking. However this is mainly due to the stress level and more importantly the type and form of waste and its pretreatment.

The variation of the shear stress (τ) with normal stress (σ) as reported by several authors at conventional strain levels of about 10 to 15 % is shown in fig. 19 and at about - 20% strain in fig. 20. It is interesting to note that some aspects are similar to the behaviour of conventional material such as soil. Indeed in the case of compacted waste bales, higher values of friction angle are attained at low normal stress levels. Whereas the interlocking is revealed under higher vertical stresses. Overall the shear angles have a low value due mainly to the presence of large amounts of plastic materials in the tested bales. In the case of old refuse, higher friction angles and intercept cohesion are obtained due to the mixed matrix of the material (soil - waste) and also due to the range of stress level. A curved linear failure envelop can be fitted through the data to account for the stress level.

Comments on shear resistance

Various authors have presented values of shear strength of waste, obtained from Laboratory, in situ tests, or from back analysis. It has become acceptable to present the reported values in terms of Mohr-Coulomb strength parameters, intercept cohesion (c) and shear angle (ϕ). However, one has to be very careful with this type of diagram. In general, the different type of tests are reported as pair of c and ϕ satisfying equilibrium. In the case of laboratory tests, the results provide an envelope in the Mohr-Coulomb plane and hence on equivalent pair of c and ϕ can be determined. However, when in-situ tests or field performance are back calculated, the infinite pairs of c and ϕ satisfying equilibrium is the result of having one known equilibrium equation, (factor of safety = 1, failure condition or ultimate load) with two unknown parameters (c and ϕ). Based on this type of analysis, one can conclude that a unique Mohr-Coulomb characterisation of waste strength from back analysis is

impossible. This viewpoint is strongly supported unless a reliable stress-strain relationship in p-q-e terms could be set.

The very wide scatter in the results observed in fig. 21 makes it difficult to draw any constructive conclusion.

A better approach to compare the various reported domestic solid waste strength data is to plot mobilised shear strength against average normal stress as shown in fig. 22 in a manner similar to the one reported by Howland & Landva (1992). The Howland and Landva (1992) approach is to plot the back calculated pairs of c and ϕ satisfying equilibrium as corresponding shear stress versus normal stress (fig. 22). The value of c calculated for $\phi = 0$ is the average shear strength mobilised along the ultimate boundary or "failure" surface (point A, fig.22). For a material with a linear strength envelope, the point of the c - ϕ pairs satisfying equilibrium (point B, fig. 22) indicates a measure for the average normal stress along the failure surface (point C, fig. 22). The crossing point (point B, fig. 22) is considered to be the one consistent datum derived from an individual case study. The crossing point from each back analysis case study is transferred to a summary plot of shear stress versus normal stress in order to develop a strength envelop for domestic solid waste. The average normal stress based on the crossing point is checked by estimating the location of the failure surface and calculating the average normal stress based on the reported unit weight of the waste and any applied loads.

The above approach has been applied to plot fig. 23. It should be borne in mind that compilation of such data is always difficult due to the lack by information and details (especially strain levels) about the case study or test. However, a somewhat curved linear failure envelope can be fitted through the obtained data, a bi-linear envelope will be assumed for the sake of simplicity. Two distinct zones can be distinguished : 1) zone A corresponding to low stress levels where the ϕ values are higher and c values very low. 2) zone B corresponding to higher stress levels. Where the failure is non frictional and c values are higher.

Based upon the above remark and the data plotted in fig. 23 an approximate estimation of the strength parameters can be made. However, as Howland & Landva (1992) pointed out it is not recommended that the strength parameters selected in fig. 23 be used by reference for analysis. It is recommended that the general methodology of comparing available directly measured and back calculated data on a mobilised shear strength versus normal stress basis be used. Finally, in other cases, and additional assumption is made, and individual values of c and ϕ are reported. The most usual additional assumptions are either that of purely cohesive ($\varphi = 0$) or purely frictional ($c = 0$) behaviour. For domestic wastes, there is no firm basis for such assumptions. As a consequence, for field tests it is better to not assume c and ϕ being zero.

Our proposal would be fig. 24 which should give approximate starting design values of c and ϕ according to three distinct zones :

- zone A corresponding to very low stresses (0 kPa $\leq \sigma_v <$ 20 kPa) where the domestic solid waste behaviour can be only cohesif. In this case, $c \approx 20$ kPa.
- zone B corresponding to low to moderate stresses (20 kPa $\leq \sigma_v <$ 60 kPa). In this case, $c = 0$ kPa and $\varphi \approx 38°$.
- zone C corresponding to higher stresses ($\sigma_v \geq 60$ kPa). In this case, $c \geq 20$ kPa and $\varphi \approx 30°$

However, the recommendations for relevant case to case testing given previously, should always be taken into account.

Limited attempts have also been made to estimate the shear strength parameters of municipal solid wastes by in-situ testing. These parameters are usually back analysed using conventional solid mechanics theory. Singh & Murphy (1990), Gifford et al. (1992) and Coumoulos et al. (1995) reported the results of vane shear and SPT that were performed at various landfills. One has to bear in mind that vane shear device is small compared with the inclusions that make up the municipal solid waste, the shear strength data obtained in this case may not be representative of full scale conditions. Such test requires also homogeneous testing material to achieve useful results. In the case of SPT there is considerable uncertainty in assigning strength parameter based on this type of test since there are no published correlations between MSW strength and blow counts.
Referring to the CPT family, the most popular and widely used in conventional text book material is nowadays still referring to the mechanical cone (CPT (M)). Following the new trends and the international standard, an increasing number of piezo (CPTU) cones are available. Engineers are interpreting daily the results in these text book materials. However,

for non text book material such as domestic waste it is much more difficult for the practising engineers to know how to use the test results in a meaningful way.
Nevertheless, cone penetration tests may, in some cases, be useful for the investigation of the waste body (for ex : to localize qualitatively areas with waste materials reducing the stability of a landfill). In this domain, Ghent University promotes for several years the use of SASW techniques in order to obtain small strain stiffness and from those values, to evaluate strength characteristics.

CONCLUSIONS

1. Besides of the difficulties in obtaining reliable values of the moisture content and relevant unit weight values for MSW, increasing attention of the todays' designers is linked to the proper assessment of the hydraulic conductivity of the waste.
2. Still too frequently, the mechanical characteristics of MSW result from estimation ; few are derived from back-analysis and only rarely real in situ or laboratory testing is coming in. Even when testing is undertaken, the interpretation of the results remains subjected to many difficulties, mainly due to the lack of conceptual reference models and clearly defined testing rules. This is one of the biggest challenges for future development in MSW characterisation.
3. In the paper, the load-settlement behaviour and some first modeling attempts are derived, trying to take into account also the biodegradation effects.
4. The shear resistance of MSW is dealing with concepts of "cohesion" intercept and shear angle, in equivalency with traditional soil mechanics. On the basis of TX on MSW, the "failure" in many cases cannot be described properly according to a simple Mohr-Coulomb approach for example ; mainly because of too large strain limits during the test. A more reliable way for discussing results can consist in plotting mobilized shear strength versus mean normal stress at well defined strain levels, for given MSW-type and age. The stress levels, the waste pre-treatment and the age of the MSW are predominant parameters in the interpretation. Some recommendations were suggested for young MSW deposits.

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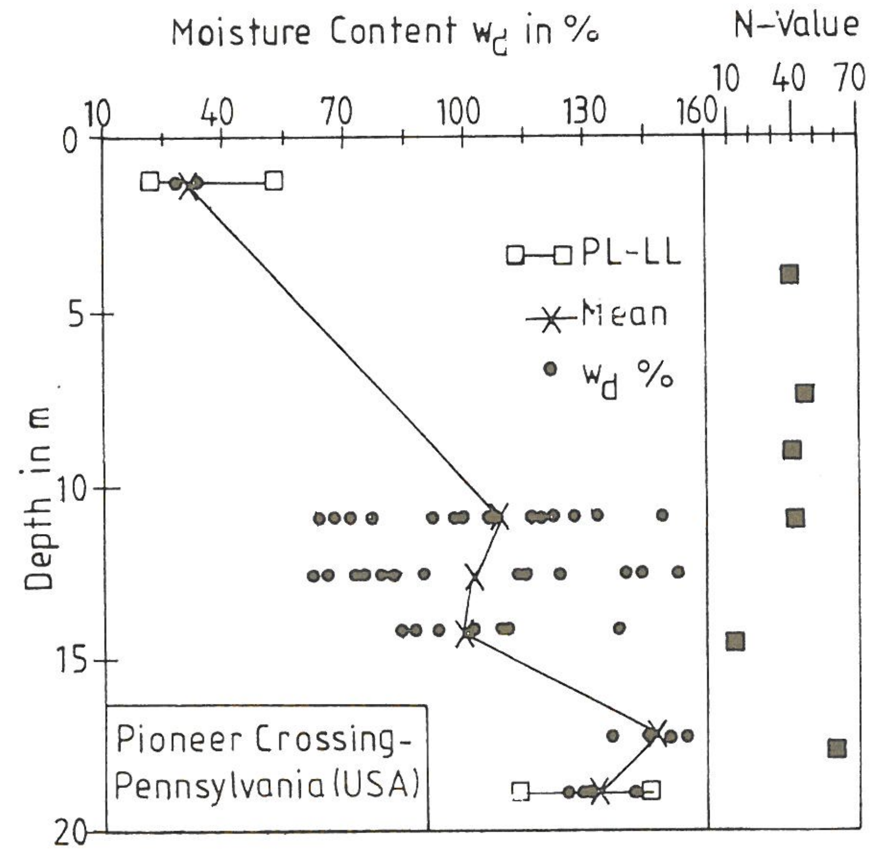


Fig. 1 : Variation of water content, liquid limits, and plastics limit with depth (Gabr & Valero 1995)

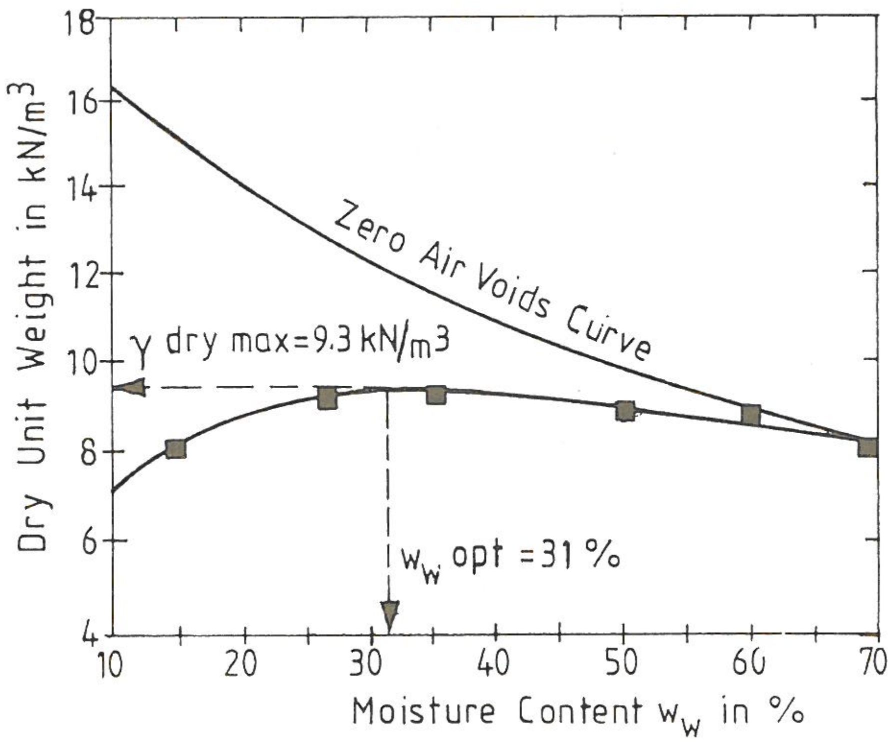


Fig. 2 : Compaction curve : maximum dry density and optimum moisture content

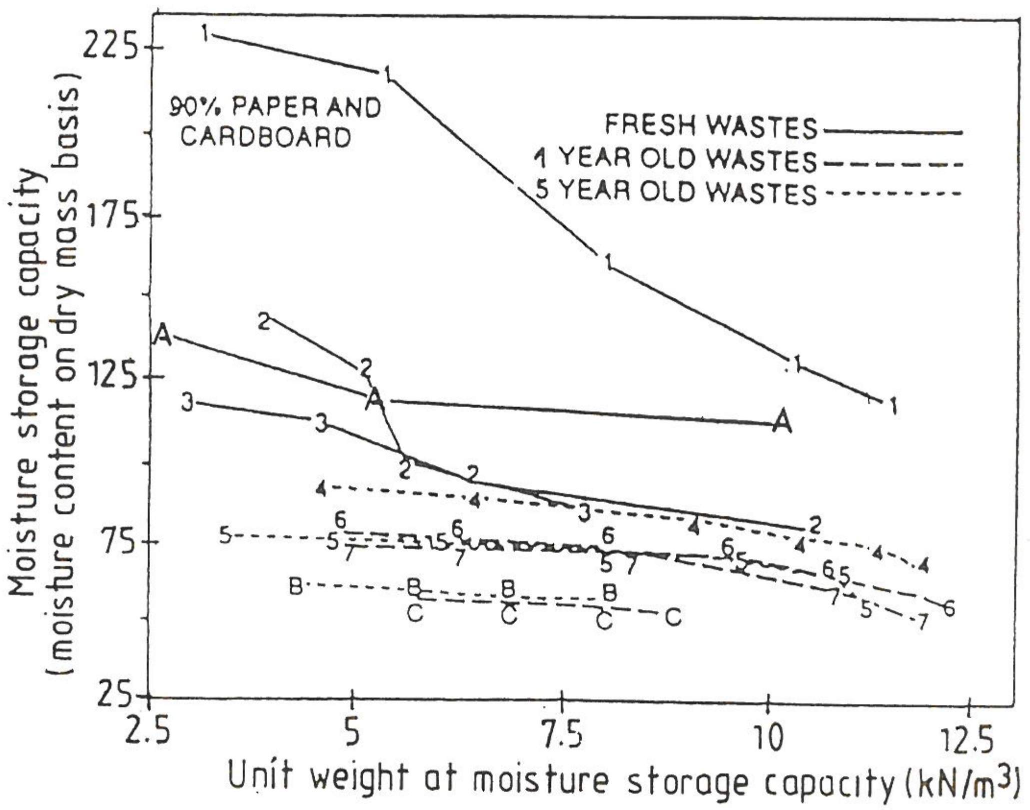


Fig. 3a : Measurement of field capacity of refuse (from Blight et al., 1992)

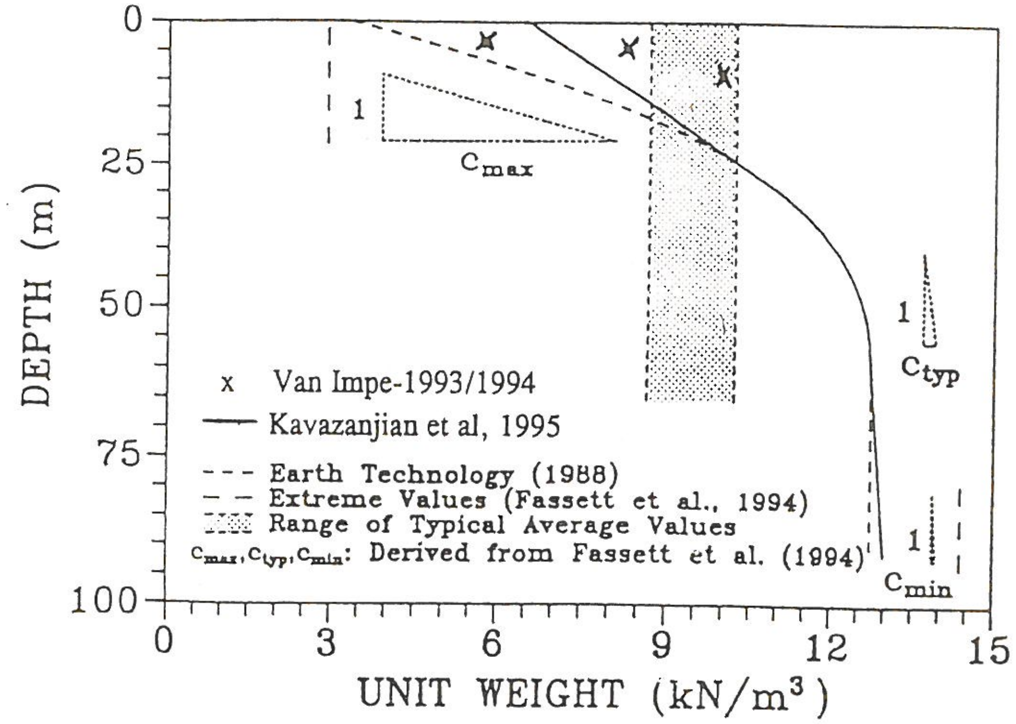


Fig. 3b : Unit Weight Profile for MSW

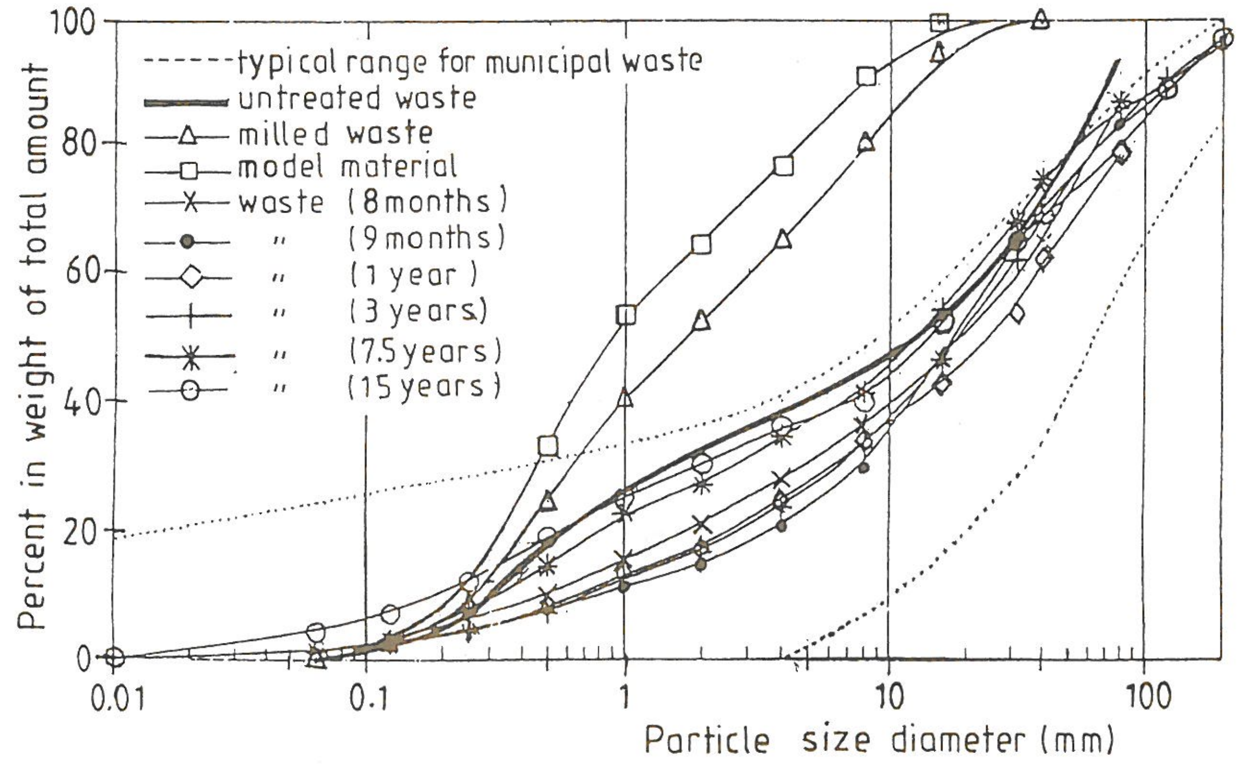


Fig. 4 : Grain size distribution of untreated milled and model municipal solid waste (from Jessberger, 1994)

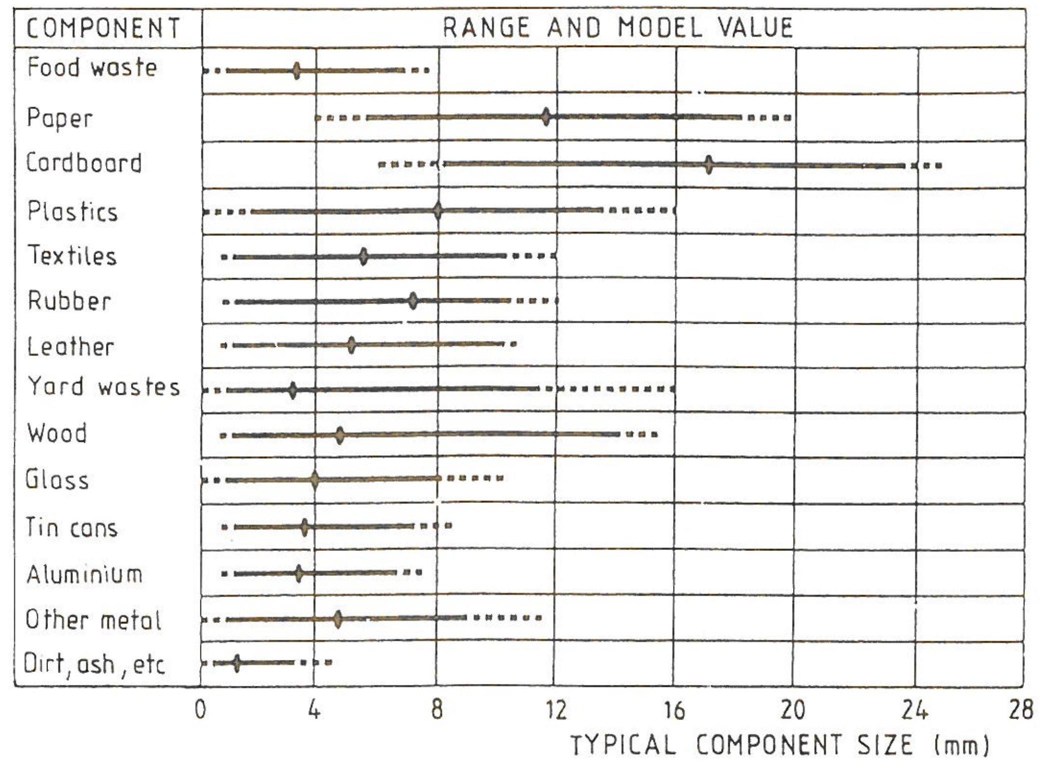


Fig. 5 : Typical size distribution of the components found in MSW (from Tchobanouglous et al., 1993)

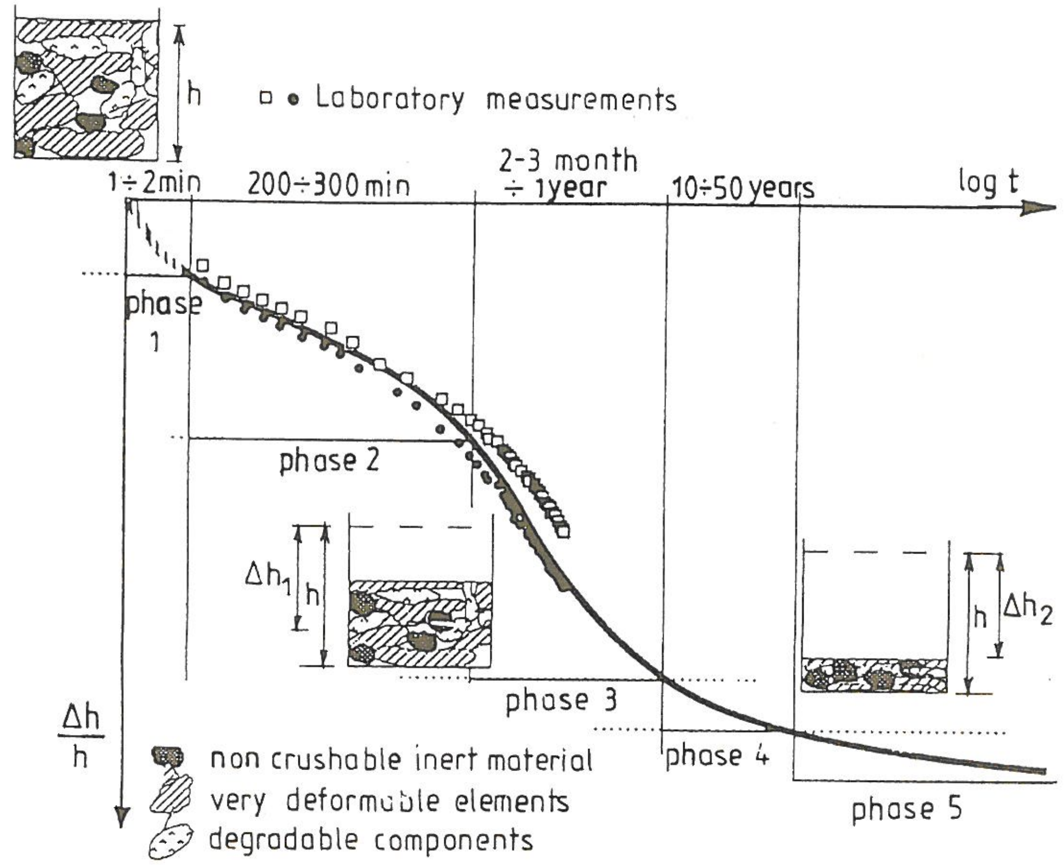


Fig. 6 : General compression curve from MSW (adapted from Grisolia et al, 1992)

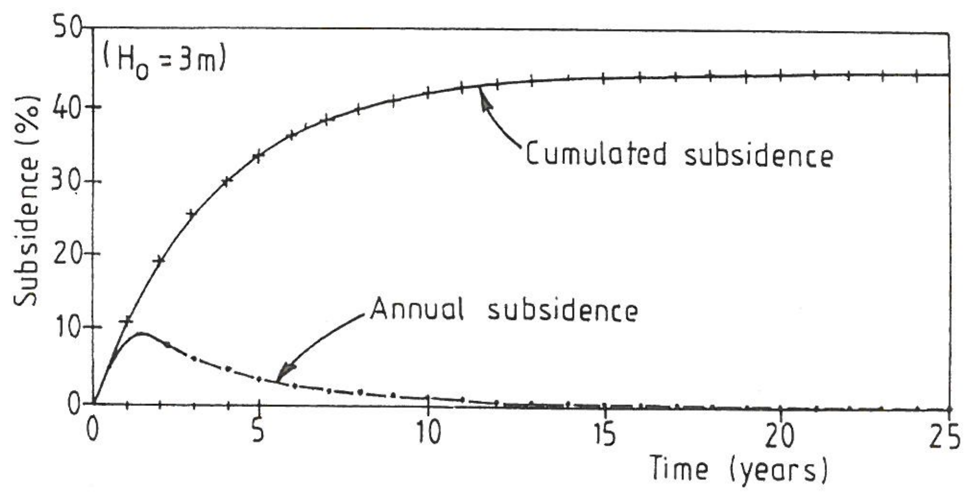


Fig. 7 : Time progression of the specific MSW subsidence in a large diameter (1 m) drained column test (after Gandolla et al, 1995)

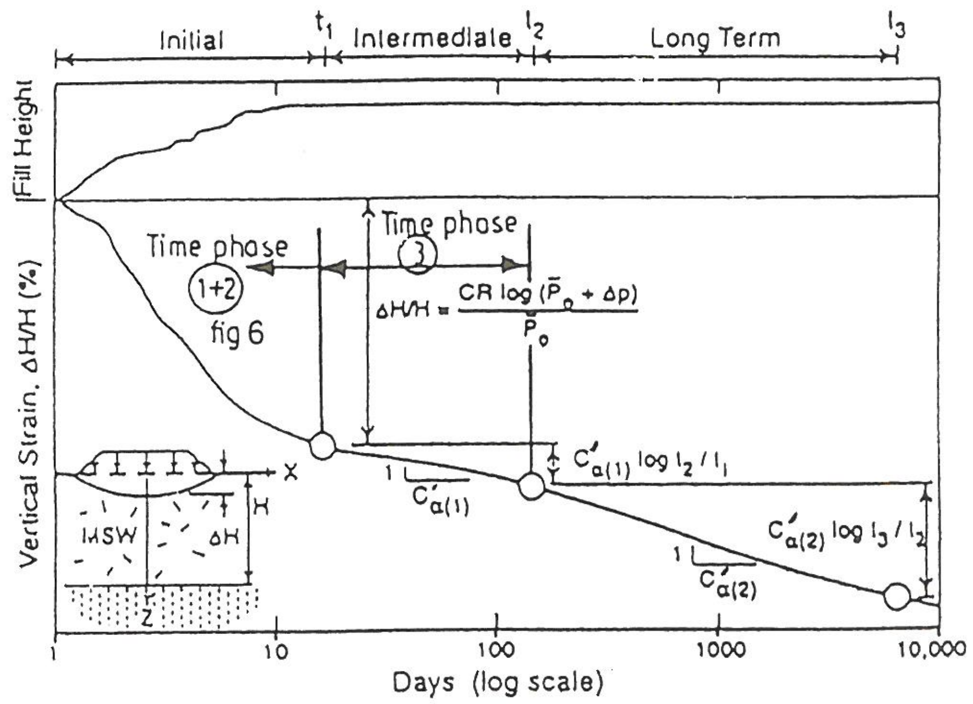
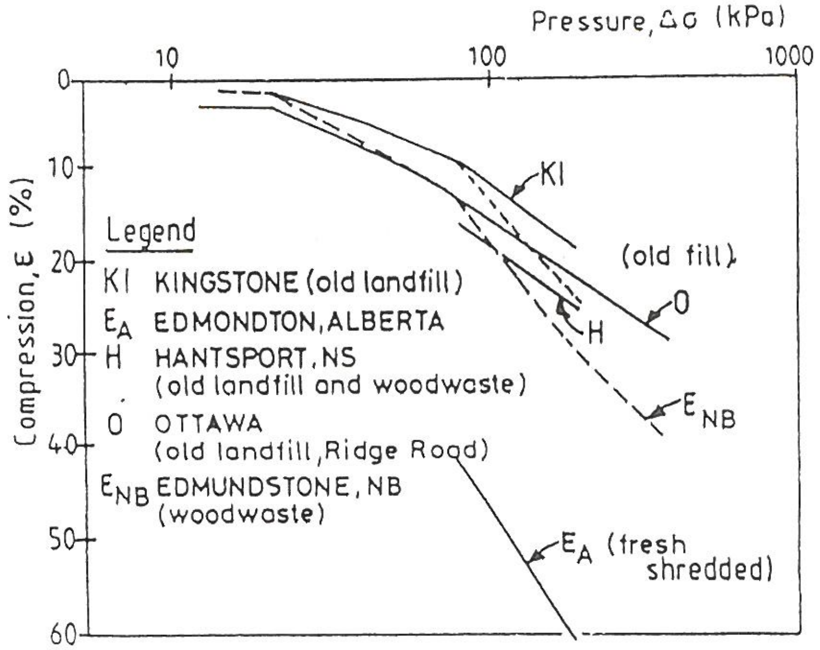


Fig. 8 : Settlement model proposed by Bjarngard & Edgers (1990)



KI	CR = 0.17	(p = 20-200 kPa)
E_A	CR = 0.35	(p = 80-200 kPa)
H	CR = 0.22	(p = 80-200 kPa)
O	CR = 0.21	(p = 100-400 kPa)
E_{NB}	CR = 0.36	(p = 100-400 kPa)

Fig. 9a: Compressive strain versus pressure for various fills (from Landva & Clark, 1990)

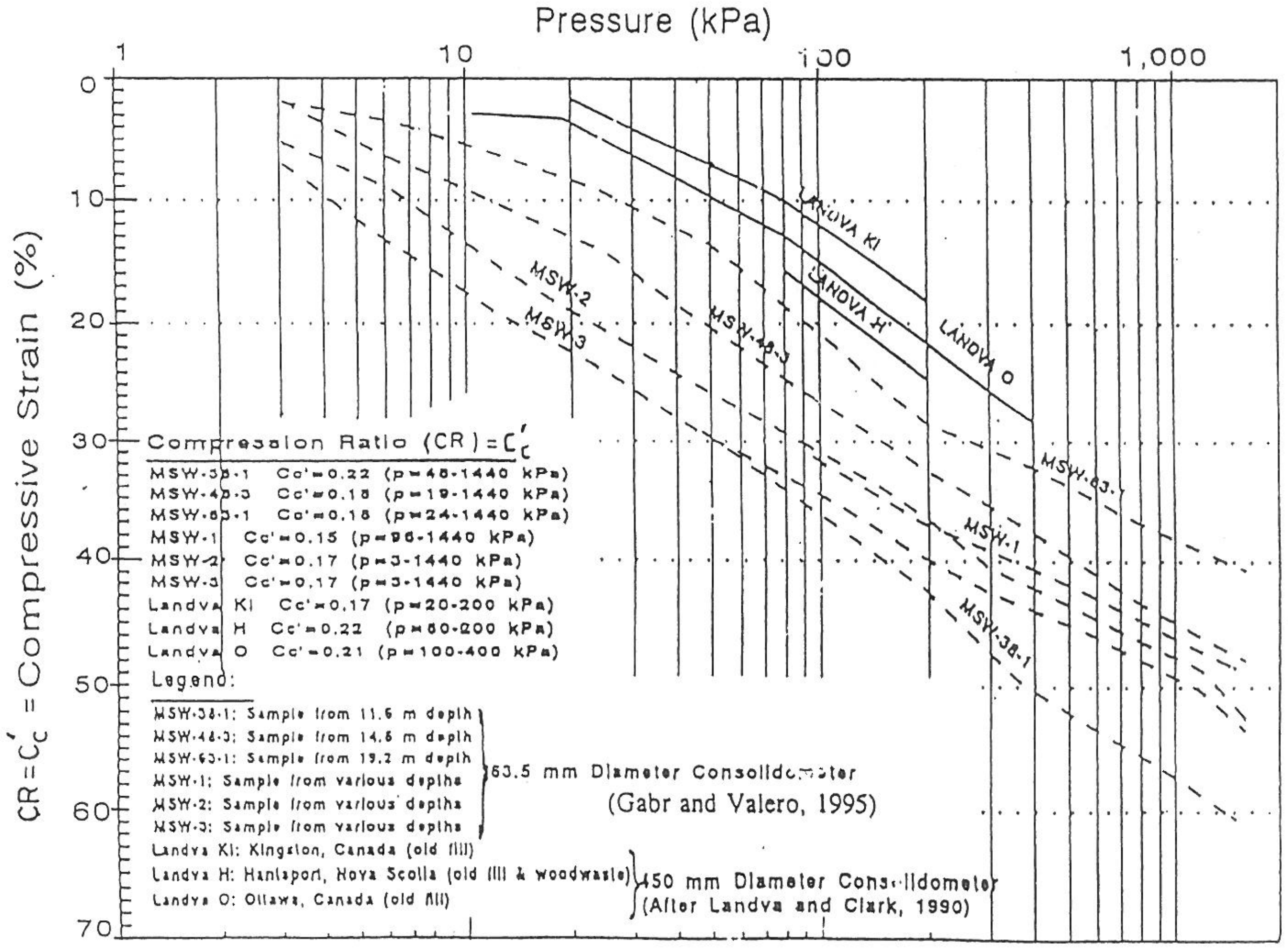


Fig. 9b : Compressibility characteristics as measured from conventional and unconventional tests

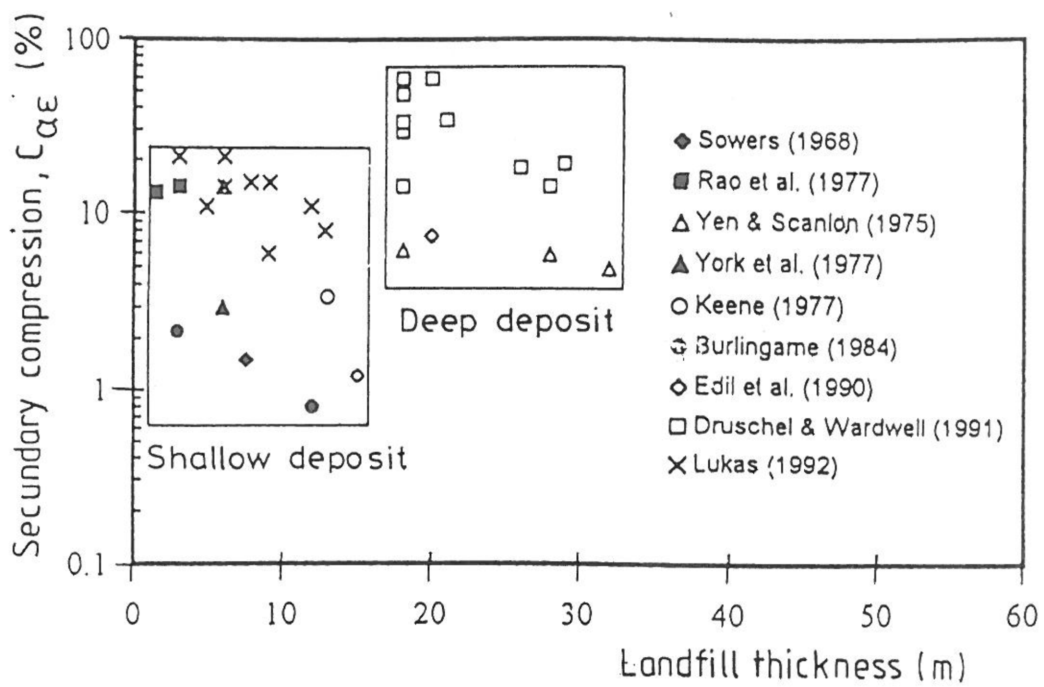


Fig. 10 : Variation of $C_{\alpha\varepsilon}$ with landfill thickness (Van Impe et al. - 1996)

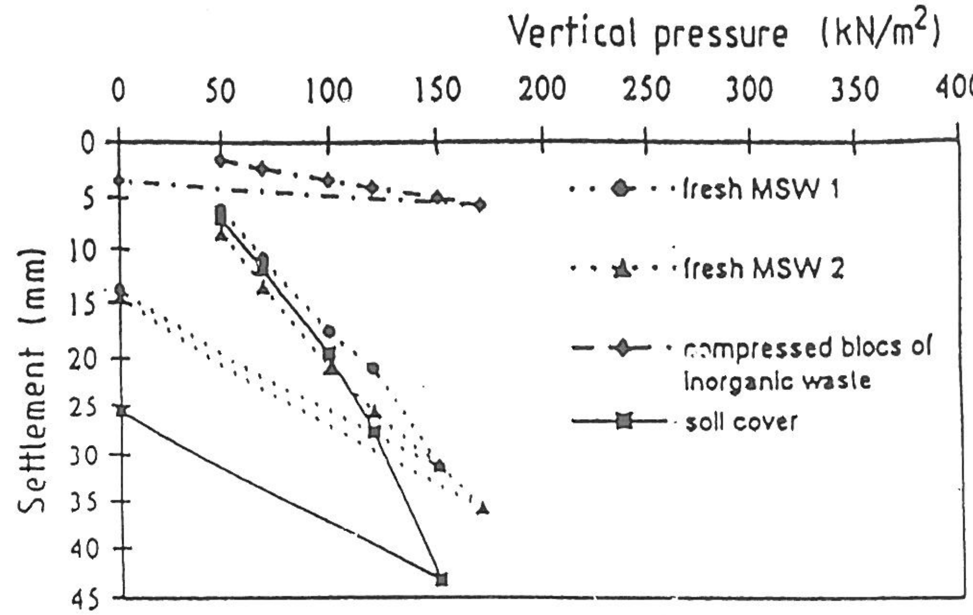


Fig. 11 : Plate load tests at the Appels landfill, Dendermonde, Belgium (W.F. Van Impe et al., 1994)

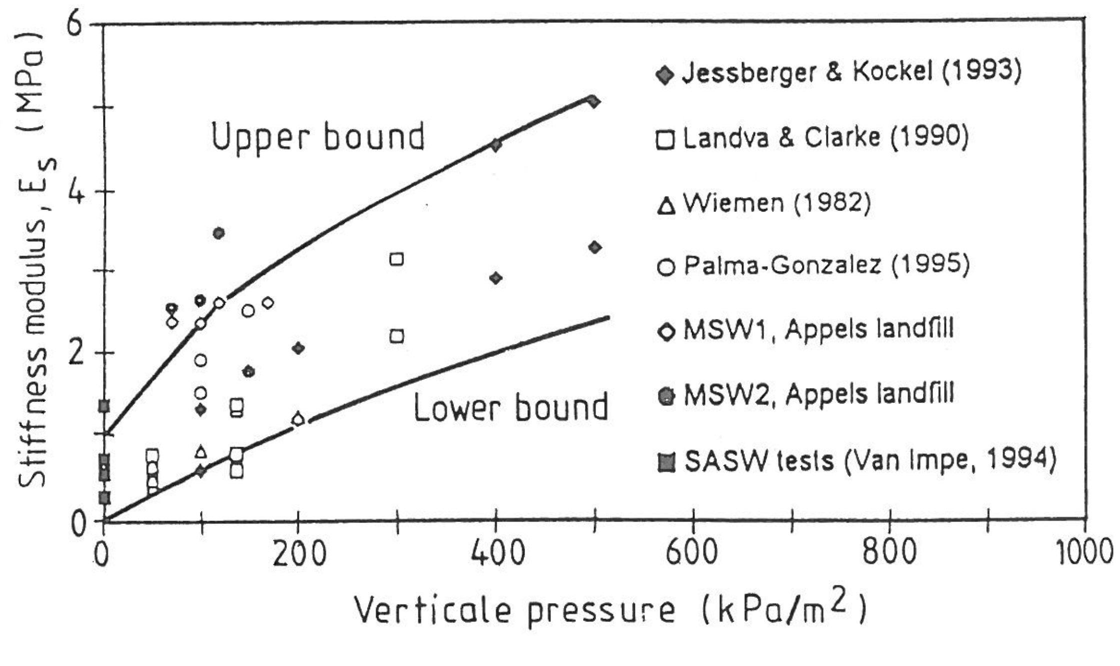


Fig. 12 : Variation of stiffness modulus with vertical stress (Van Impe et al. - 1996)

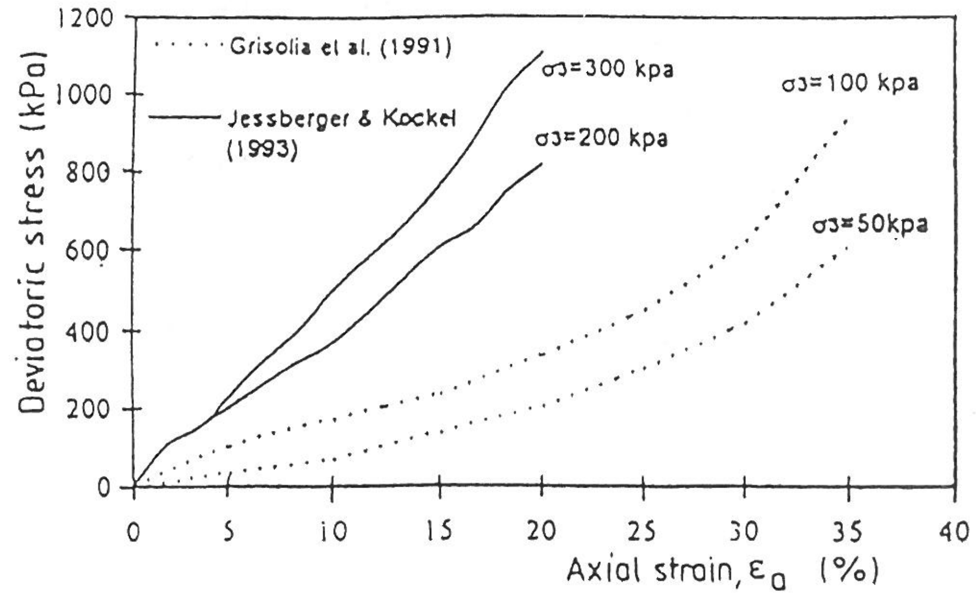


Fig. 13 : Typical stress-strain relationship for MSW

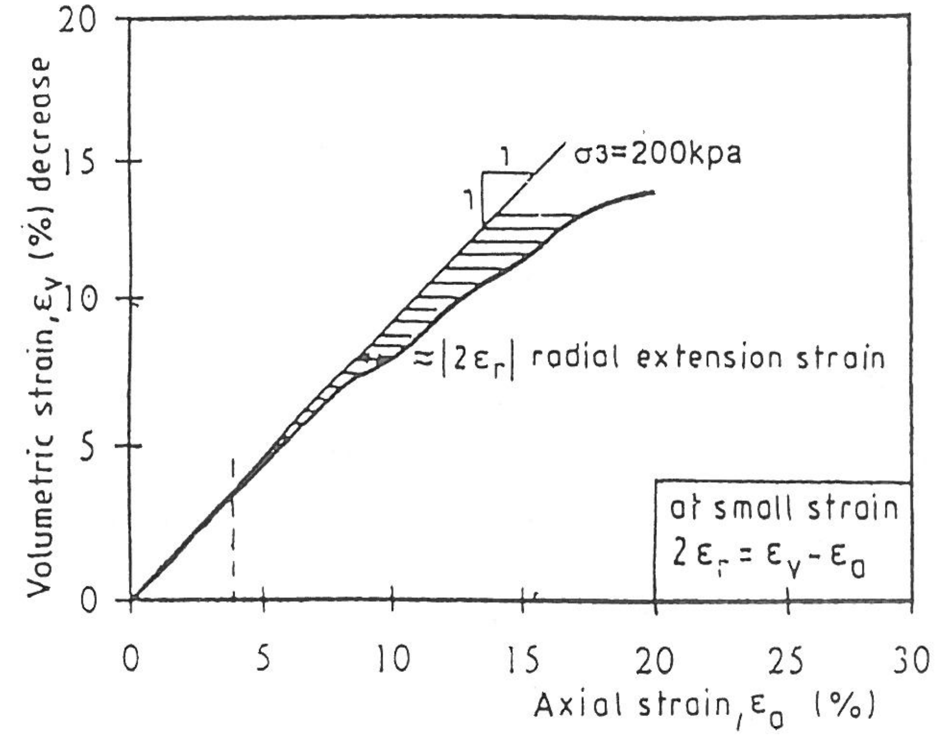


Fig. 14 : Variation of volumetric strain with axial strain for MSW
(from Jessberger & Kockel, 1993)

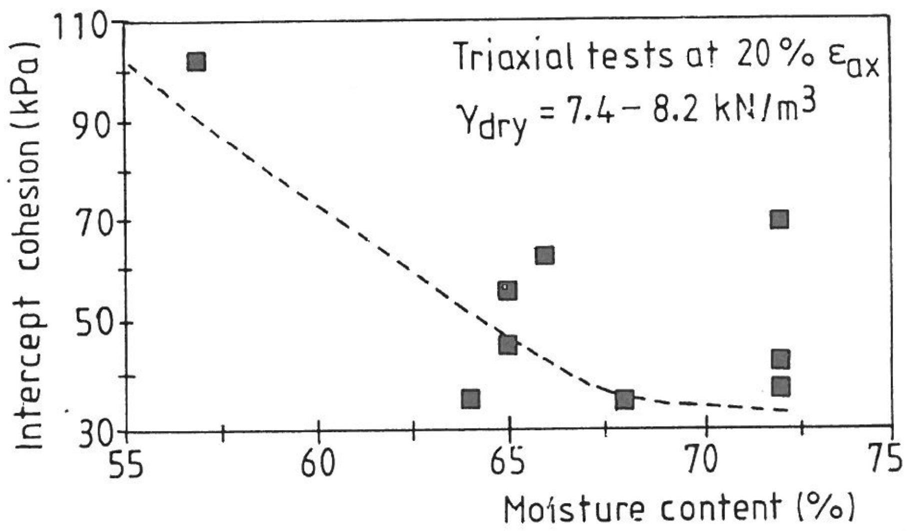


Fig. 15 : Variation of intercept cohesion as a function of moisture content
(after Gabr et al, 1995)

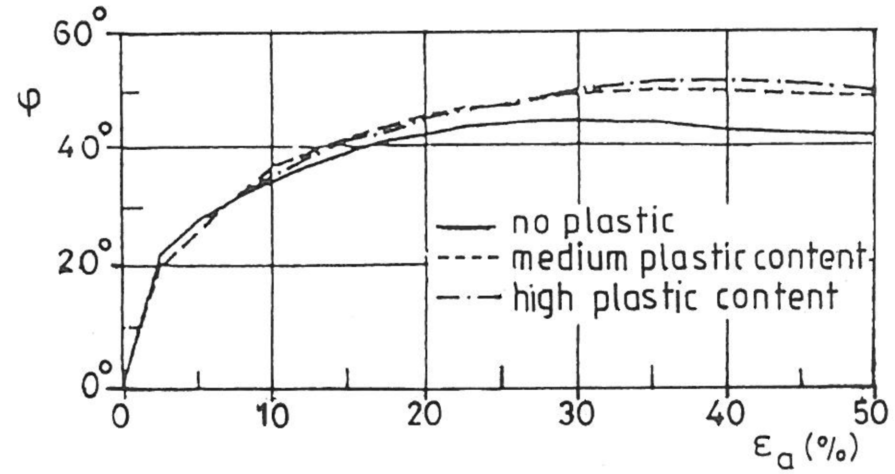


Fig. 16 : Deformation dependent activation of friction angle
(from Kockel & Jessberger, 1995)

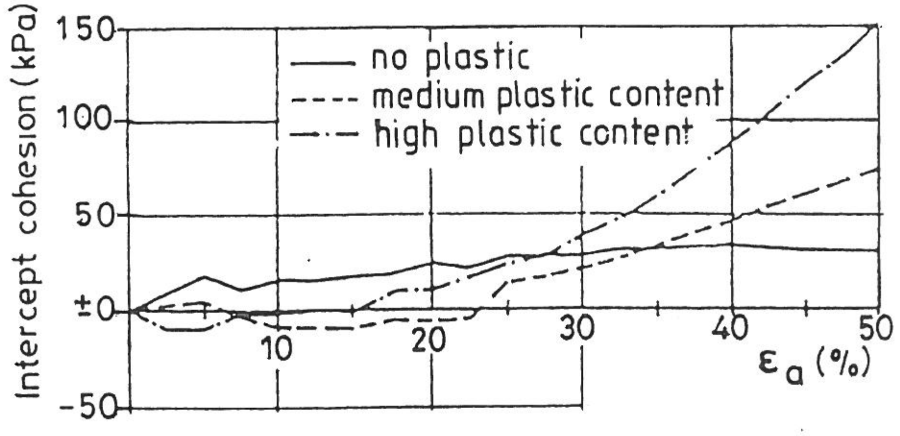


Fig. 17 : Deformation dependent activation of intercept cohesion
(from Kockel & Jessberger, 1995)

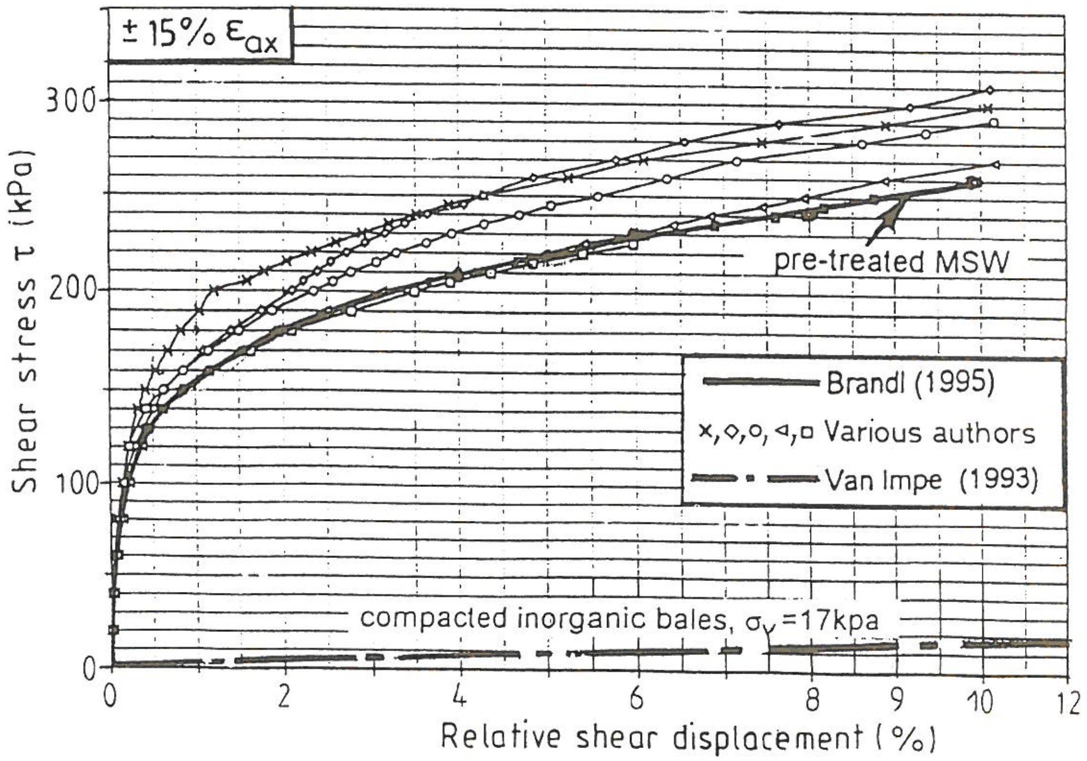


Fig. 18 : Stress strain relationship obtained from direct shear tests

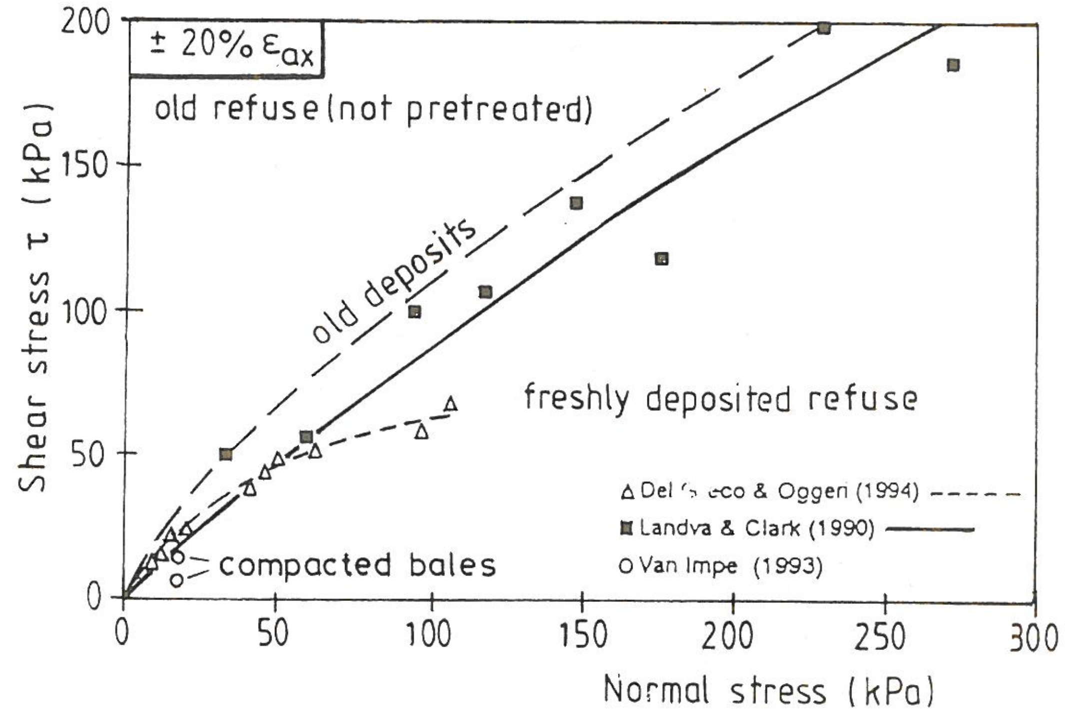


Fig. 19 : Shear-stress relationship for MSW (from direct shear tests)

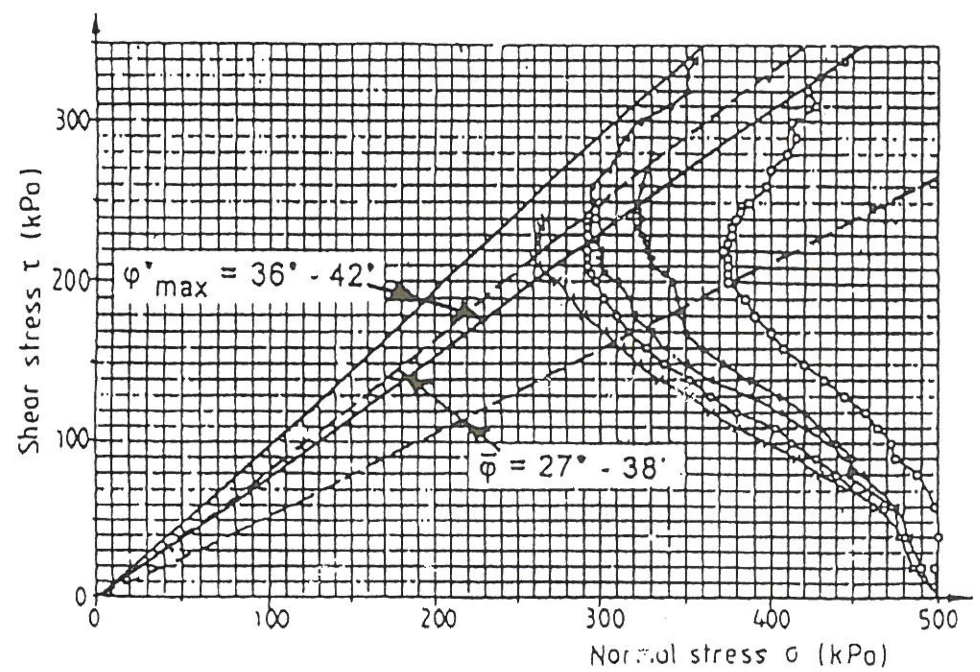


Fig. 20 : Stress paths of shear tests with constant void ratio for pretreated municipal waste (with glass splinters). Scattering of fictitious friction angle, φ^*_{max} (for shear displacement $\gamma \geq 20$ %) and of shear angle, φ, in the transition to loosening (cfr. Brandl - 1995)

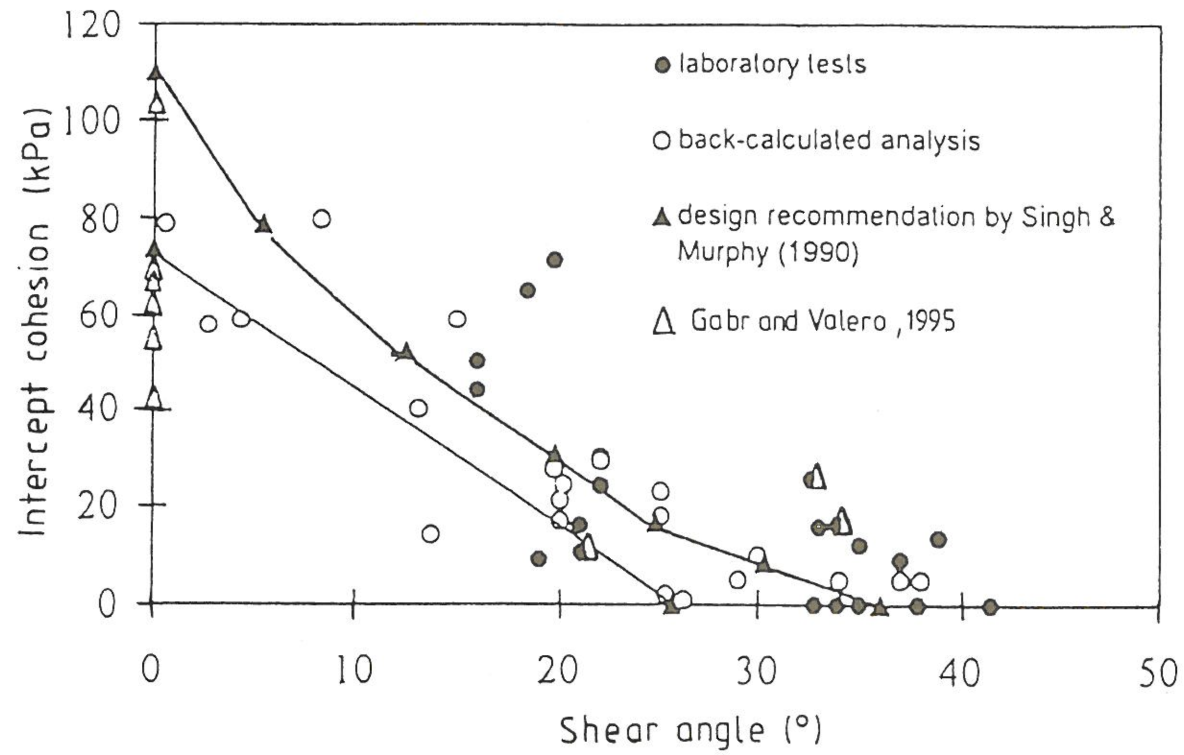


Fig. 21 : Strength parameters of MSW estimated with different methods

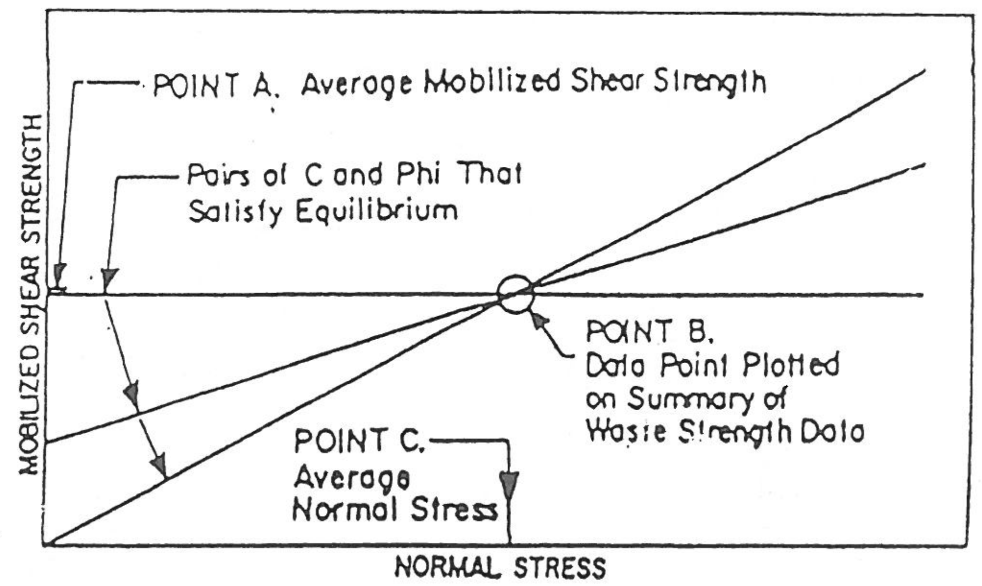


Fig. 22 : Method of analysing an individual waste strength case history (from Howland & Landva, 1992)

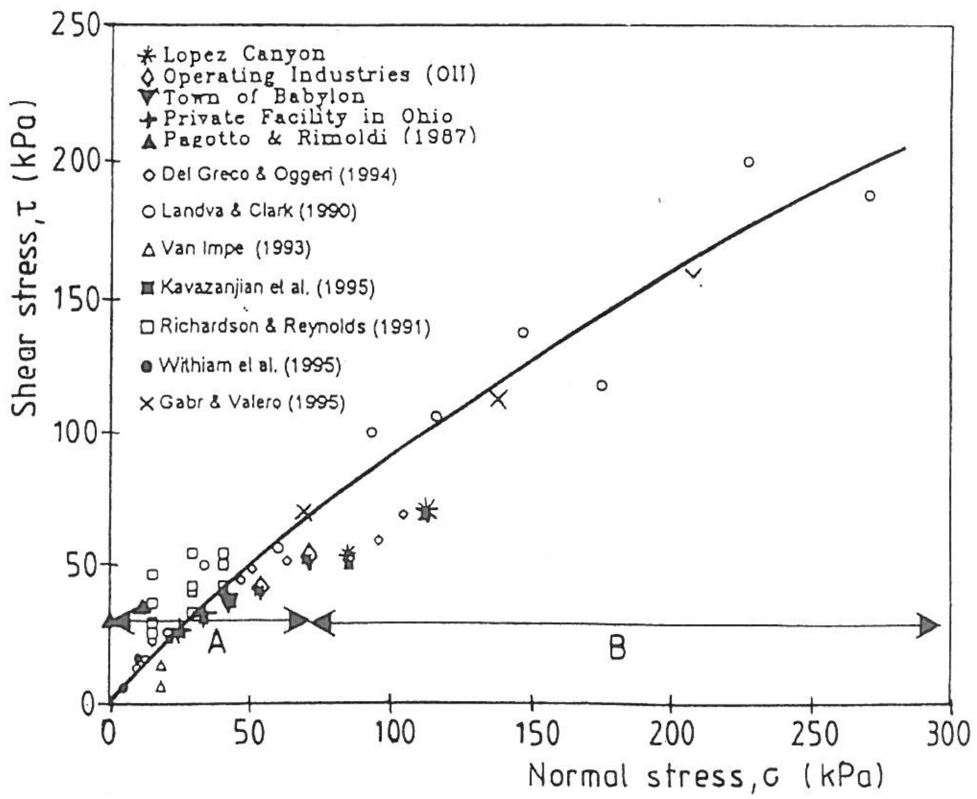


Fig. 23 : Domestic solid waste strength data from various sources (Van Impe et al. - 1996)

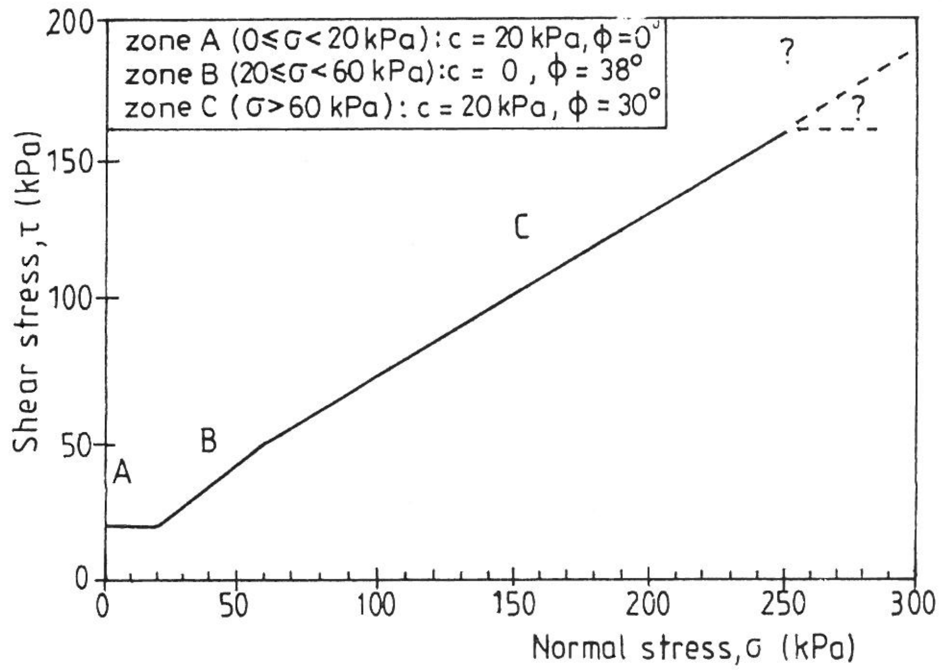


Fig. 24 : Domestic solid waste strength data design recommendation (Van Impe et al. - 1996)