



TRINITY COLLEGE DUBLIN
COLÁISTE NA TRÍONÓIDE

**THE
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Soil Plasticity Determination Using Manafi Method and Apparatus

MSc by Research in Geotechnical Engineering

Seyed Masoud Manafi Ghorabaei

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Declaration

I declare that this thesis has not been submitted as an exercise for a degree at this or any other university and it is entirely my own work.

Seyed Masoud Manafi Ghorabaei, February 2017

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Abstract

Determination of soil plasticity is of great importance in geotechnical engineering projects. Consistency of soil can be expressed in terms of Atterberg limits of soils. Liquid limit and plastic limit tests are among the two most regular tests for determination of index properties of fine grain soils. Soil classification of fine grain soils is based on results of Atterberg limit tests. In addition, many engineering properties of soils can be correlated to the results of soil plasticity determination tests. Current standard methods for determination of soil plasticity have several issues that make their results unreliable in many cases. In this research, it is tried to address problems of conventional methods regarding soil plasticity determination, design a new apparatus for determination of liquid limit and plastic limit of soils, and standardise a new test method for determination of soil plasticity with more accuracy and reliability in comparison to conventional methods. Soil consistency is a qualitative phenomenon. In this regard, author's suggestion for solving the problems related to the determination of soil plasticity is utilisation of qualitative research approach. Accordingly, the author has proposed a new test method and designed a new apparatus (Manafi Apparatus) for determination of soil consistency based on the nature of soil deformation utilising qualitative research approach, referred to as Manafi Method. In this technique, it is possible to determine the workability of soil at Atterberg limits and specify the consistency state of the soil. Ten different soils were selected carefully to cover a wide range of soils plasticity with plastic limits range from 18.64% to 30.78%, and the liquid limits range from 30.25% to 61.77%. The most differences between the results obtained by proposed method and standard methods were 2.38% and 1.94% of water content for liquid and plastic limits correspondingly. The experimental results obtained by designed apparatus confirmed the proposed method and provided more consistent results in soil plasticity determination in comparison to current standard methods (fall cone and thread rolling methods).

Keywords: Plasticity, Liquid Limit, Plastic Limit, Atterberg Limit Tests, Soil Consistency, Manafi Method, Manafi Apparatus, Casagrande Percussion Cup Method, Fall Cone Method, Thread Rolling Method

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Notations

Abbreviations

ASTM	American Society for Testing and Materials
BS	British Standard
CBR	California Bearing Ratio
CH	Clay With High Plasticity
CL	Clay With Low Plasticity
EP	Extrusion Pressure
ER	Extrusion Ratio
<i>LI</i>	Liquidity Index
<i>LL</i>	Liquid Limit
MH	Silt With High Plasticity
ML	Silt With Low Plasticity
MPM	Mud Press Machine
<i>PI</i>	Plasticity Index
<i>PL</i>	Plastic Limit
<i>PL</i> ₁₀₀	Water content of a soil with an undrained strength 100 times that at the liquid limit of that soil
SC	Clayey Sand
SM	Silty Sand
SD	Standard Deviation
<i>SL</i>	Shrinkage Limit
SMP	Soil Mini-Penetrometer
SPT	Standard Penetration Test

Symbols

$\dot{\bar{\epsilon}}$	Mean effective strain rate
\bar{P}	Average power
\bar{v}	Average speed of an object
v_0	Initial velocity of soil specimen
$\bar{\epsilon}$	Natural (effective) strain
$\bar{\sigma}$	Flow stress
τ_f	Frictional stress
A	The final area of deformed material
a	Average acceleration of soil specimen
A_0	Initial area of deformed material
A_c	Cross-sectional area of container
A_E	Cross-sectional area of extruded material
A_R	Real contact area
c_u	Undrained shear strength

D	Diameter of billet
D_C	Inside diameter of the container
D_E	Equivalent diameter of the extruded material
F	Magnitude of force
F_D	Force required for deformation of soil
F_E	Extrusion force
F_F	Friction force
F_p	Force applied by the press
F_r	Required extrusion force
F_s	Side wall friction between soil material and container wall
k	Material shear strength
l	Final length of deformed material
L	Billet length
L'	Die bearing length of a solid die
l_0	Initial length of deformed material
m	Friction factor between billet and container interface
m	Mass of soil specimen
m'	Friction factor between flowing material and die-bearing interface
m''	Factor of friction between die bearing and extruded material
N	Normal force
n	Number of symmetrical holes in container
P_D	Pressure required for the plastic deformation of the material
P_F	Pressure required to overcome frictional resistances
pH	The 'p' stands for the mathematical power, and the 'H' for hydrogen ions
P_R	Pressure to overcome redundant or internal deformation work
p_r	Radial pressure
P_s	Inner pressure (specific pressure)
P_T	Total extrusion pressure
R^2	Pearson product moment correlation coefficient
T	Material temperature
t	Time of extrusion
V	Average ram speed
V_a	Volume of air
V_d	Volume of dry soil
V_E	Extrusion speed
V_R	Ram speed
V_s	Volume of solids
V_w	Volume of water
W	Work was done on an object
W_D	Work was done for deformation of soil
w_i	Water content at liquid limit

w_p	Water content at plastic limit
w_s	Water content at shrinkage limit
W_s	Weight of soil specimen
Z_1	Billet butt thickness
α	Angle of dead material zone
Δt	Time interval
Δx	Magnitude of displacement
θ	Angle between directions of \vec{F} and $\Delta\vec{x}$
μ	Friction coefficient
σ	Stress

1 Introduction

The most important soil aggregate property of cohesive soils is the consistency, while the relative density is the most important soil aggregate properties of granular soils (Terzaghi et al. 1996; Murthy 2003). Water plays a great role in soil cohesion. Hence, water content is an important factor in aggregate behaviour of cohesive soils. Consistency is a term which is used to indicate the degree of firmness of cohesive soils which is expressed qualitatively by terms such as very soft, soft, stiff, very stiff, extremely stiff, and hard. The reason is that the physical and engineering properties of cohesive soils considerably varies with different water contents. Cohesive soils with high water content may be very soft but become harder if they lose their moisture content gradually. As it is evident, it is required to define several borders for these qualitative terms to identify the consistency state of cohesive soils.

Albert Mauritz Atterberg (1846–1916), a Swedish chemist, observed that cohesive soil behaves more like a solid at a very low moisture content, but when it gradually absorbs water, becomes a slurry and may flow like a liquid. Hence, he categorised the behaviour of soil, depending on the moisture content, into several states and introduced seven limits of the behaviour of fine-grained soils at varying water contents (Atterberg 1911). Later, Karl Terzaghi (1926) applied Atterberg's method for preliminary classification of soils in geotechnical engineering and after that extended by his research assistant Arthur Casagrande. Since then, many studies have been done on determining Atterberg limits with different methods. However, a precise, reliable, applicable, popular, and accurate enough method for different types of soils has not been established yet.

Among seven different limits introduced by Atterberg (1911), only three main limits of shrinkage limit (*SL*), plastic limit (*PL*), and liquid limit (*LL*) have remained in regular usage in geotechnical engineering. According to these limits, four basic states are classified as: solid, semisolid, plastic, and liquid. Therefore, consistency of a soil can be expressed in terms of Atterberg limits of soils. Accordingly, the water contents pertinent to the transitional behaviour from one state to another are termed as Atterberg Limits, and limit state determination tests are Atterberg Limit Tests.

Atterberg limit tests are among the most usual tests in geotechnical engineering. Their usage in soil classification systems is one of their most important applications (Casagrande 1932; Casagrande 1948; Casagrande 1958). In addition, they are obtainable easily for large number of samples and their results provide quick assessment and correlations to important physical and engineering properties of soil (Skempton 1954; Skempton 1957; Black 1962; Heukelom and Klomp 1962; Schofield and Wroth

1968; Stroud 1974; Whyte 1982; Nagaraj and Murthy 1985; Nakase et al. 1988; FHWA-IF-02-034 2002; Kayabali 2012b; Vardanega and Haigh 2014). *PL* and *LL* that outline the plastic behaviour of soil is of great concern for geotechnical engineers which is the subject of this research. Plasticity index (*PI*) is the range of water content that soil behaves plastically and is the difference between water content of liquid limit and water content of plastic limit (ASTM D4318-10e1 2010; ASTM D653-14 2014). Two well-known standards for the determination of liquid and plastic limits are BS EN 1377-2 (1990) and ASTM D4318-10e1 (2010) that describe the tests procedures in details. By determination of *LL* and *PL* of soils, it is possible to determine the range of water content that the soil behaves plastically. However, accurate and reliable determination of these limit states by different methods are still in dispute.

1.1 Aims and Objectives

This research has three main objectives:

- 1- Addressing problems of conventional and alternative methods regarding soil plasticity determination.
- 2- Designing a new apparatus for determination of liquid limit and plastic limit of soils.
- 3- Standardising a new test method for determination of soil plasticity with more accuracy and reliability in comparison to conventional methods.

1.2 Thesis Structure Outline

The thesis is presented in five chapters:

- Chapter 1 is related to the introduction of the research project.
- Chapter 2 explains the previous research done on the plasticity determination tests with a critique on the uncertainties and deficiencies of conventional methods. Some alternative apparatus and methods proposed by researchers are discussed as well. In addition, necessity of proposing new methods and apparatus for a better soil plasticity determination is explained.
- Chapter 3 describes the Manafi apparatus and its properties. It will also include the procedure of experiment and the typical test results. The progress of the apparatus evolution is also covered in this chapter.
- Chapter 4 covers the experimental research conducted by Manafi apparatus along with data analysis. In addition, inaccuracy of current standard methods is quantified in this chapter.
- Chapter 5 summarises the results of the research and recommends future investigations pertinent to soil plasticity determination.

2 Literature Review

2.1 Introduction

Various properties of soils are usually related to geometry of soil particles (i.e. size and shape of particles), and consistency or relative density of soils. These properties can be expressed as index properties of soils and it is generally possible to be studied under two main categories (Terzaghi et al. 1996; Murthy 2003):

1. Soil grain properties;
2. Soil aggregate properties.

The geometry of soil grains and morphological/mineralogical aspects of finer fractions of soil are among the main soil grain properties. The most important soil aggregate property of cohesive soils is the consistency, while the relative density is the most important soil aggregate property of granular soils (Terzaghi et al. 1996; Murthy 2003). Water plays a great role in soils cohesion. Hence, water content is an important factor in the aggregate behaviour of cohesive soils. However, soil aggregate properties of granular soils do not change considerably by water content fluctuation. In other words, gravitational forces govern the engineering characteristics of coarse-grained particles (Murthy 2003), which are considered coarser than 75 μm , whereas interparticle forces are predominant in fine-grained soils which are finer than 75 μm according to ASTM D422-63 (2002).

Fine grain soils behave differently when they have various water contents. Consistency is a term which is used to indicate the degree of firmness of cohesive soils which is expressed qualitatively by terms such as very soft, soft, stiff, very stiff, extremely stiff, and hard. The reason is that the physical and engineering properties of cohesive soils considerably varies with different water contents. Cohesive soils with high water content may be very soft but become harder if they lose their moisture content gradually. As it is evident, it is required to define several borders for these qualitative terms to identify the exact consistency state of the cohesive soil.

Albert Mauritz Atterberg (1846–1916), a Swedish chemist who devoted his time to soil studies at the end of 19th century, observed that cohesive soil behaves more like a solid at a very low moisture content, but when it gradually absorbs water, it becomes a material like a slurry and may flow like a liquid. Hence, he categorised the behaviour of soil, depending on the water content, into several states and introduced seven limits of the behaviour of fine-grained soils at varying water contents (Atterberg 1911). Later, Karl Terzaghi (1926) applied Atterberg's method for preliminary classification of soils and after that extended by his research assistant Arthur Casagrande. Since then, many

studies have been done on determining Atterberg limits with different approaches. However, a precise, reliable, applicable, popular, and accurate enough method for different types of soils have not been established yet.

Casagrande (1932) names 6 most important of consistency limits in his report which earlier were established by Atterberg:

1. The upper limit of viscous flow, at which a mixture of clay and water flows as a fluid almost like water.
2. The lower limit of viscous flow, the "liquid limit" (LL), at which two sections of a soil cake, placed in a cup barely touch but do not flow together under the impact of several sharp blows.
3. The "sticky limit", at which the clay loses its adhesive property and ceases to stick to other objects, such as the hands or a metal blade.
4. The "cohesion limit" (Zusammenhaftbarkeits-grenze), at which the grains cease to cohere to one another.
5. The lower limit of the plastic state, or the "plastic limit" (PL), at which the soil crumbles when being rolled out into threads.
6. The lower limit of volume change or the "shrinkage limit", at which further loss of moisture does not cause a loss in volume.

Among these limits, only three main limits of shrinkage limit, plastic limit, and liquid limit have remained in regular usage. According to these limits, four basic states are recognisable namely: solid, semisolid, plastic, and liquid which are shown in Figure 2-1. Therefore, consistency of a soil can be expressed in terms of Atterberg limits of soils as it is demonstrated in Figure 2-2. The water contents pertinent to the transitional behaviour from one state to another are termed as Atterberg Limits and the tests required to determine the limits are the Atterberg Limit Tests.

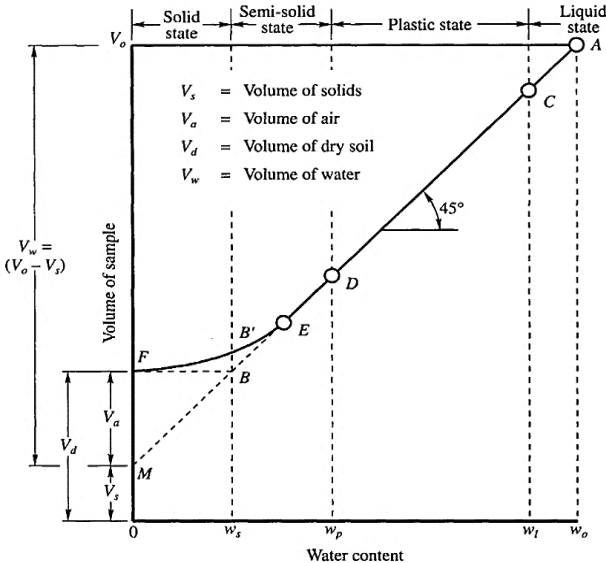


Figure 2-1: Extract from Murthy (2003). Curve showing transition stages from liquid to solid state

States	Limit	Consistency
Liquid		Very soft
..... w_l	Liquid limit	Soft
Plastic		Stiff
..... w_p	Plastic limit	Very stiff
Semi solid		
..... w_s	Shrinkage limit	Extremely stiff
Solid		Hard

Figure 2-2: Extract from Murthy (2003). Different states and consistency of soils with Atterberg limits

Atterberg limit tests are among the most regular tests in geotechnical engineering. Their usage in soil classification systems is their most important application. In addition, they are obtainable easily for a large number of samples and their results provide a quick assessment of important physical and engineering properties of soil. Plastic limit and liquid limit that outline the plastic behaviour of soil is of great concern for geotechnical engineers which is the subject of this project. Plasticity index (PI) is the range of water content that soil behaves plastically and is the difference between water content at the liquid limit (LL) and water content at the plastic limit (PL) (ASTM D4318-10e1 2010).

Atterberg limit tests are very important and they are also called indexed tests because their results provide key information related to strength, permeability, compressibility, and shrink/swell potential of soil (FHWA-IF-02-034 2002). It is also possible to correlate the results of Atterberg limit tests to a ratio of the strength of Standard Penetration Test (SPT) blow count and California Bearing Ratio (CBR) of soils (Vardanega and Haigh 2014). Plastic behaviour of soil is a criterion for soil classification systems in geotechnical engineering. Therefore, Atterberg tests do not only apply for clay soils, but also are performed for silts, organic soils and peat, silty and clayey sands to find out whether they are SM or SC based on the unified soil classification system. Plasticity index of soil is also correlated to internal friction angle, over-consolidation ratio, and coefficient of lateral earth pressure of soils (Kayabali 2012b). All of these important properties of soil, and much more can be estimated by conducting several simple, quick, and inexpensive Atterberg limit tests. Therefore, it is critical to making sure that the results obtained by these tests are reliable.

2.2 Standard methods for determination of liquid limit

One of the main approaches for determination of liquid limit is the strength based approach. In this approach, it is assumed that all soils have a particular small shear strength at the liquid limit. Casagrande percussion cup method is a test that has been accepted as a method of liquid limit determination of soils in many standards around the world including ASTM and BS standards. This test determines the dynamic slope stability

of soil and relates the liquid limit to a specific ratio of soil strength to soil density (Haigh 2012). In this method, it is tried to find a particular water content at which the solid particles lose contact with one another and the soil begins to flow and behaves as a liquid material (Vardanega and Haigh 2014).

Another popular test method for determining liquid limit is Fall Cone Test which was devised and developed in Sweden in 1915 (Whyte 1982). This method measures shear strength of soil directly (Vardanega and Haigh 2014) and correlates the liquid limit of cohesive soils to a specific shear strength for all soils. Different cone sizes had been tested in many studies (e.g., Terzaghi 1927; Sowers et al. 1959; Karlsson 1961; Sherwood and Ryley 1970; Garneau and LeBihan 1977; Littleton and Farmilo 1977; Wroth and Wood 1978; Houlsby 1982; Wood 1982; Wood 1983; Wood 1985; Wasti and Bezirci 1986; Leroueil and Le Bihan 1996; Feng 2001) and several standard procedures are decided in different national standards (e.g., CAN/BNQ 2501-092-M-86 1986; BS EN 1377-2 1990; SS 027120 1990).

Two well-known standards for the determination of liquid limit are BS EN 1377-2 (1990) and ASTM D4318-10e1 (2010) that describe the tests procedures in details. Key points of the methods in these standards will be briefly discussed in this document.

2.2.1 Meaning of liquid limit

It is very important to consider the main definition of liquid limit in the determination of soil plasticity. The liquid limit in cohesive soils is defined as “the water content, in percent, of a soil at the arbitrarily defined boundary representing the transition from the semi-liquid to plastic states” (ASTM D653-14 2014). In BS EN 1377-2 (1990), the liquid limit is defined as “the empirically established moisture content at which a soil passes from the liquid state to the plastic state”. As it is evident, the word “arbitrarily” and “empirically” in liquid limit definitions are because of quantifying a qualitative phenomenon and incapability of current standard methods for accurate and consistent determination of these states in various soil types.

2.2.2 Sample preparation

There are 3 ways of sample preparation in BS EN 1377-2 (1990) and ASTM D4318-10e1 (2010) standards. Using of natural soil wherever possible is recommended by BS EN 1377-2 (1990). In this method, coarse particles greater than 425 microns exist in the sample will be removed by hand or with tweezers. It is important to notice that drying, even air drying, might affect the plasticity properties of some soils significantly (Fookes 1997). Hence, the soil should not be dried before testing and whenever it is required, the method used should be stated in the report (BS EN 1377-2 1990).

Whenever usage of natural soil is not possible, it is recommended to prepare samples with wet sieving method (BS EN 1377-2 1990). In this method slurry of soil will be passed through a 425 μm sieve and finishing by washing with distilled or deionized water until it runs clear. The washings passing the sieve will be protected and will be partially dried until a stiff paste will be obtained that can be used in liquid limit tests (BS EN 1377-2 1990; ASTM D4318-10e1 2010).

Another sample preparation method which is not recognised by BS EN 1377-2 (1990) for definitive plasticity tests, but is documented in ASTM D4318-10e1 (2010) is dry preparation method. In this method, the specimen will be dried at room temperature or in an oven with less than 60 $^{\circ}\text{C}$ until it is possible to pulverise the soil clods. The material will be passed through a 425 μm sieve and the soil remained on the sieve will be soaked in a small amount of water and again will be passed through a 425 μm sieve. The obtained slurry will be added to the dried material previously sieved through the 425 μm sieve.

Water plays a great role in soils consistency. It is very important to not use tap water for washing soil and preparing the soil paste. It is possible that the cations of salts in tap water exchange with the natural cations in the soil and greatly affect the test results (ASTM D4318-10e1 2010). It is recommended to use distilled water for the test by ASTM standard. Furthermore, demineralized water is also permitted to use for the tests. In general, ASTM standard does not let to use any water with more than 100 mg/L of dissolved solids for either the soaking or washing operations (ASTM D4318-10e1 2010). BS standard also permits usage of distilled or deionized water in condition that the water has non-volatile residue (Not more than 5 mg of total dissolved solids per litre of water according to clause 8 of BS EN 1377-3 (1990)) and its pH value should not be lower than 5.0 nor higher than 7.5 (BS EN 1377-1 1990).

It is useful to have a look at different categories of water surrounding cohesive particles. There are many different factors (i.e. geometry, chemical composition, and etc.) involving the capability of cohesive soil to maintain water between its particles. Various categories of water around a clay particle is simplified and illustrated in Figure 2-3 (Head 2006):

1. Adsorbed water is a thin layer of water that attraction between negative charge of soil particle surface and positive end of water molecules provides a great force that changes the physical properties of water.
2. Hygroscopic moisture which is a layer of water around clay particle that for its removing it is required to heat the soil and use the oven drying.
3. Capillary water, which is under surface tension and it is removable by air drying generally.
4. Gravitational water, which moves in the voids of soil grains and it is possible to remove it by a drainage system.

5. Chemically combined water, which is a kind of hydration water inside crystal structure of clay particle and is not removable by oven drying in some kinds of clays.

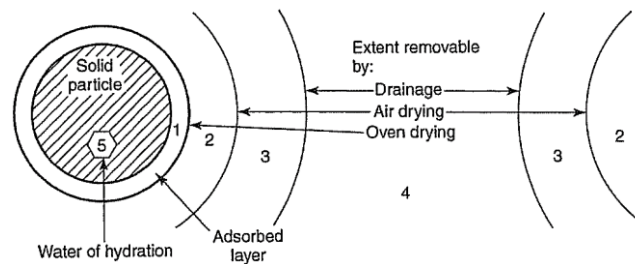


Figure 2-3: Extract from Head (2006). Various categories of water surrounding clay particles

Adsorbed water plays a great role in soil consistency. It is also stated that the plasticity of soil is due to the adsorbed water that surrounds the clay particles (Das 2009). The adsorbed film of water might also be available around coarse particles, however, it is much thinner in comparison with the diameter of the particles due to the scarcity of negative charge on the surface of coarse-grained soil particles. In contrast, fine-grained soils and especially clays adsorbed water is relatively much thicker and might even become bigger than the grain size (Murthy 2003).

The higher the specific surface of the soil, the more negative charge on the surface of soil grains, hence the more forces related to the adsorbed layers. These forces are very high that alters the mechanical and physical properties of water like a solid within the vicinity of soil particle. It is required to heat the clay particle to more than 200 °C to evaporate the adsorbed water which shows the great bond between the surface of clay particle and water molecules (Murthy 2003). The more distance from soil particle surface, the less viscous water surrounding soil particles. Therefore, adsorbed water plays a great role in physical properties of cohesive soils, but have little effect on granular soils. Beyond the influence zone of adsorbed water, there will be remaining mentioned waters that are possible to be removed by standard oven temperature of 110 ± 5 °C as specified in ASTM D2216-10 (2010) or 105 - 110 °C in BS EN 1377-2 (1990).

In addition, Atterberg limit test results can also significantly affected by chemistry and pH of any water which is added to the soil sample for preparing the homogeneous soil paste (Davidson 1983; Hobbs 1986; Yang and Dykes 2006; Asadi et al. 2011; O’Kelly 2016). Chemical condition of soil and water system affects remoulded response of soils too (Rosenqvist 1953; Bjerrum 1954; Penner 1965; Torrance 1975; Torrance and Pirnat 1984). The pH of water affects cation exchange capacity of soils. Surface of cohesive soil minerals has a negative electrical charge that enables them to absorb cations and polar compounds. Hence, even usage of distilled water can provide different results with

what it might happen in the field. Torrance and Pirnat (1984) investigated the effect of pH on the rheology of a marine clay by changing the pH of the material. They showed that the remoulded shear strength of soil was pH-dependent as it is demonstrated in Figure 2-4. Ideally, the water used for increasing moisture content should not be distilled water. In fact, the water should be obtained from the site of soil sample at the sample's depth if applicable as the chemical properties might be very variable at different depths (i.e. different pH values in a bog profile shown in Figure 2-5). However, this might be impractical for many cases.

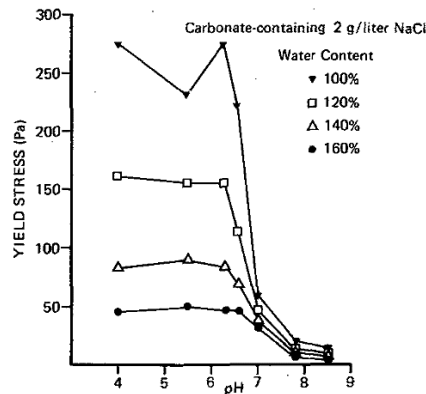


Figure 2-4: Extract from Torrance and Pirnat (1984). Yield stress-pH relationships at constant water content for a marine clay

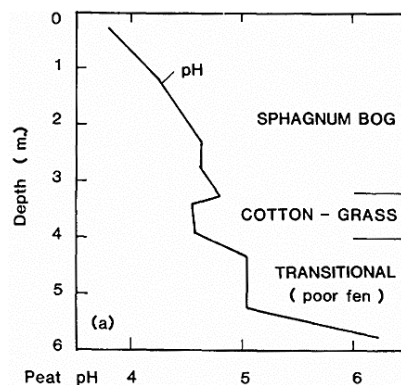


Figure 2-5: Extract from Hobbs (1986). pH in a bog profile

2.2.3 Casagrande (percussion cup) method

Atterberg (1911) suggested a procedure of liquid limit determination based on visual observation of soil stability of a groove in clays. In this technique, a soil paste would be grooved in a cup, and the cup was struck on hand, and the stability of the groove would be observed (Vardanega and Haigh 2014). Lots of uncertainties were involved in this method and it was very dependent on the person who had done the test. Later, Casagrande (1932) tried to standardise this method and devised a new apparatus for determination of the liquid limit state of cohesive soils which nowadays is known by Casagrande (percussion cup) method and is the standard method of liquid limit determination recognised by ASTM standard (ASTM D4318-10e1 2010). This method is

also recognised as a standard method for determination of the liquid limit state of cohesive soils in BS standard (BS EN 1377-2 1990). However, it is suggested to use fall cone method due to fewer uncertainties involved in its procedure compared to Casagrande method in addition to difficult maintenance of Casagrande apparatus and operator's judgment which affect the test results (BS EN 1377-2 1990).

In this method, liquid limit of soil will be determined by doing several trials of the test. In the test, some matured and homogeneous specimen is put in a brass cup and is spread. Then, it is divided in two half by a special grooving tool that is designed to produce a groove with a somewhat defined geometry. After that, the cup is dropped from a particular height for several times with a fairly constant rate. Because of the shocks and induced energy, the two halves merge together and the groove closes within a specific length. There are two methods for determination of liquid limit in Casagrande test: multipoint test, and one-point test (ASTM D4318-10e1 2010). In multipoint test method, at least three trials within a specific range of drops with varying water content are required to provide the data for plotting or calculating the liquid limit water content. In one-point test method, two trials at one water content within a specific range of drops are required to provide the data for calculating the liquid limit water content using a correction factor. It is recommended to use multipoint test method due to its greater precision in comparison to the one-point test method. In addition, one-point test method requires a skilful operator judgement, hence it is not recommended for inexperienced operators (ASTM D4318-10e1 2010). In general, the one-point method will be used in condition that there is a small amount of soil sample, or when a less accurate result is acceptable (Head 2006).

It should be noticed that it is assumed that the undrained shear strength of soil at the liquid limit state would be approximately 2 kPa (ASTM D4318-10e1 2010). The results of Casagrande test is correlated to a strength divided by density (Haigh 2012; Vardanega and Haigh 2014). Actually, Casagrande test is a dynamic slope stability test that tries to move the slopes and make them unstable by applying vertical acceleration pulses (Wroth 1979).

2.2.4 Fall cone method

Another popular test method for determining liquid limit is Fall Cone Test (or cone penetrometer method) which had been devised and developed in Sweden since 1915 (Whyte 1982). Karlsson (1977) stated that 10 mm penetration of a 60 g Swedish cone with a 60° apex angle is a representation of 1.7 kPa shear strength for all soil types. The results obtained by this apparatus had a good correlation to hard-base Casagrande percussion cup (Whyte 1982). BS EN 1377 (1975) used an 80 g cone representing shear

strength around 1.6 kPa correlated to soft-base Casagrande percussion cup (Whyte 1982). Different cone sizes had been tested and finally a standard procedure is decided which is described in BS EN 1377-2 (1990). Due to the fewer uncertainties related to the determination of liquid limit by this method, Eurocode 7 suggests that fall cone test should be preferred to the Casagrande percussion cup (CEN. E.N. 1997-2 2007). Shimobe (2010) had also reviewed different cone penetration standards in various countries where this method has been used for liquid limit determination. According to BS EN 1377-2 (1990), if a cone with particular specifications penetrates 20 mm into the soil paste in 5 seconds, that soil would be in its liquid limit. Like Casagrande method, there are two methods for determination of liquid limit in fall cone test: definitive method, and one-point method. In the definitive method, at least four data within a suggested range is required to plot the relationship between moisture content and cone penetration which will lead to finding the particular moisture content of 20 mm cone penetration. Clayton and Jukes (1978) suggested a less elaborate method (one-point method) for determination of soil liquid limit utilising cone penetrometer test. Although the results obtained by this method is less reliable than definitive method, it is a useful method for determination of soil liquid limit when a small portion of soil is available or when a less accurate result is acceptable.

This method measures shear strength of soil directly (Vardanega and Haigh 2014) and correlates the liquid limit of cohesive soils to a specific shear strength for all soils. Wroth and Wood (1978) studied the literature for soils strengths at liquid limit state including results obtained by Skempton and Northey (1952) and Youssef et al. (1965) that are shown in Figure 2-6 and Figure 2-7 respectively.

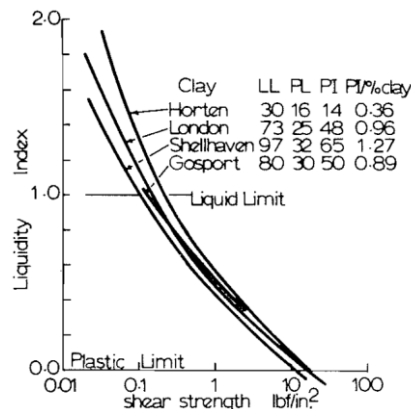


Figure 2-6: Extract from Wroth and Wood (1978). Relation between shear strength and liquidity index of remoulded clays (after Skempton and Northey (1952))

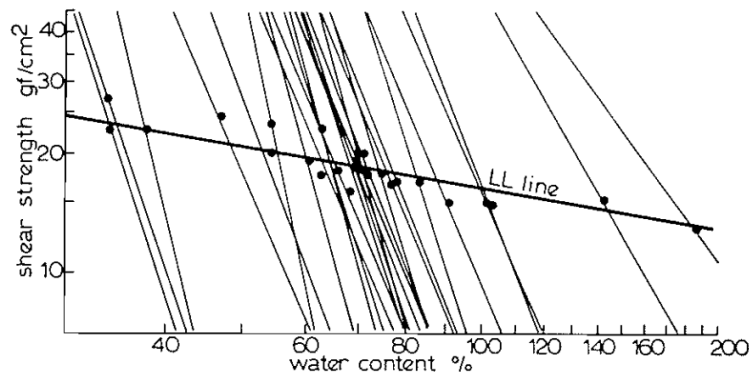


Figure 2-7: Extract from Wroth and Wood (1978). Relation between water content and shear strength. Each line represents a different soil. The solid points indicate the shear strengths at the liquid limit (after Youssef et al. (1965))

Wroth and Wood (1978) assumed that the shear strength obtained by laboratory vane test is the same as the results obtainable by the triaxial compression test and considered other available data that were in the range of 0.7 to 2.65 kPa. They considered the median of this range (1.7 kPa) as the best estimate of the undrained shear strength of soils at their liquid limit state.

Koumoto and Houlsby (2001) had described mechanics and theories of fall cone test in their paper. They had mentioned the angle of cone tip, cone sharpness, roughness of cone surface, and rate of shear strain during penetration are among the most important factors affecting the fall cone penetration results.

2.2.5 Comparison between Casagrande method and fall cone method

In general, more consistent results will be obtained by fall cone method compare to Casagrande method due to fewer experimental and operator errors involved in the procedure of the test (Sherwood and Ryley 1968). BS EN 1377-2 (1990) prefers fall cone method for determination of soil liquid limit since it is essentially a static test that depends on soil shear strength. Many researchers had reported different values of liquid limits using Casagrande percussion cup and fall cone method (Wasti and Bezirci 1986; Leroueil and Le Bihan 1996; Sridharan and Prakash 2000; Prakash and Sridharan 2006). Medhat and Whyte (1986) presented liquid limit tests values obtained by different methods for three soils demonstrated in Table 2-1. Littleton and Farmilo (1977) observed small differences between the results of Casagrande method and fall cone method for soils with liquid limits less than 100%. However, Casagrande method provided higher liquid limit values for soils with liquid limits more than 100%. Figure 2-8 shows the relationship between the results obtained by Casagrande method and fall cone method based on available data in 1978.

Table 2-1: Liquid limit tests comparison (Medhat and Whyte 1986)

Soil	BS cone test	Percussion test				Range
		ASTM		BS 1377		
		(a)	(b)	(a)	(b)	
A	33	29	31	31	33	29-33
B	45	40	43	44	46	40-46
C	59	53	56	59	63	53-63

*(a) with ASTM grooving tool, (b) with BS grooving tool.

*Values are in per cent.

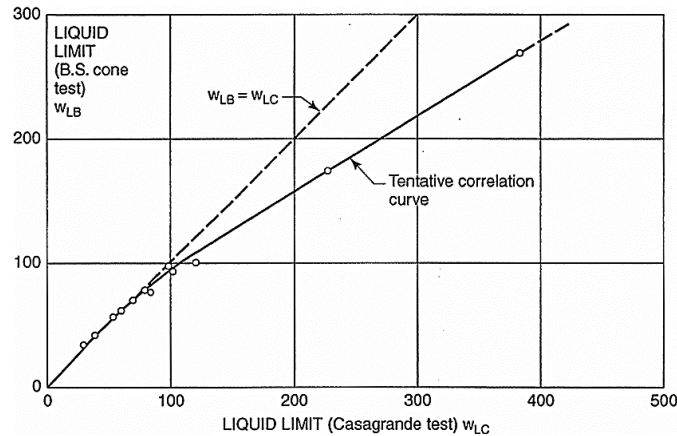


Figure 2-8: Extract from Head (2006). Correlation of liquid limit results from Casagrande and fall cone methods

Having a comparison between the results of these two popular test methods (Casagrande percussion cup and fall cone method) reveals that liquid limit values obtained by these methods are not identical. Different fundamental mechanics of these apparatus explains the deviation between the results (Vardanega and Haigh 2014).

There is a huge discussion among the researchers about the preference of each method for determination of liquid limits of soils. Sridharan and Prakash (1999) believe and provide data that fall cone method measures remoulded shear strength of soils and it is not able to measure undrained cohesion which is related to the plasticity of soil. They explained two components of the undrained strength of a soil (undrained cohesion and undrained frictional resistance). The undrained cohesion component is due to viscous diffuse double layer water around soil particles and undrained frictional resistance is because of soil fabric which is the result of the net interparticle attractive forces (Prakash and Sridharan 2006). Fall cone method determines the undrained strength mostly due to undrained friction which cannot be representative of plasticity property of soil (Prakash and Sridharan 2006). Since both of these methods are among the standard methods for *LL* determination (ASTM D4318-10e1 (2010) only recognises percussion cup method, BS EN 1377-2 (1990) recognises both of these methods). Hence, it is necessary to consider results of both of the methods for soil plasticity determination.

2.3 Standard method for determination of plastic limit

One of the main approaches for determination of plastic limit is the measurement of workability of soil by a visual method. Thread rolling method is the most popular method for determination of plastic limit of soils.

2.3.1 Meaning of plastic limit

It is essential to have a clear definition of plastic limit in the determination of soil plasticity. The plastic limit in cohesive soils is defined as “the water content, in percent, of a soil at the boundary representing the transition from the plastic to semi-solid states” (ASTM D653-14 2014). BS EN 1377-2 (1990) defines the plastic limit as: “the empirically established moisture content at which a soil becomes too dry to be plastic”. The BS definition is vague because the phrase “too dry” is very ambiguous. It is useful to have a look at the meaning of “*definition*” in Oxford Dictionary of English: “a statement of the exact meaning of a word”, “an exact statement or description of the nature, scope or meaning of something”, and “the degree of distinctness in outline of an object” (Stevenson 2010). Hence, the BS definition is not appropriate to be considered as a proper definition. However, ASTM’s definition is not perfect either. Like liquid limit definition, the plastic limit definition requires an adjective of “arbitrarily” in its meaning to cover the quantifying process of a qualitative phenomenon as long as it is determined imprecisely.

2.3.2 Thread rolling method

The procedure of this method has not changed significantly since its introduction by Atterberg in 1911 (Vardanega and Haigh 2014). In this method, it is tried to roll a portion of same soil specimen used for liquid limit and make a thread of soil that it would crumble at a specific diameter of soil thread. Detailed procedure of this method is explained in ASTM and British standards (BS EN 1377-2 1990; ASTM D4318-10e1 2010). Although the test procedure seems simple and easy, the soil is under a very complex stress and strain systems. The soil thread is under a combination of bar rolling distortion, cylinder compression and lateral extrusion processes which is a function of many factors such as amount of applied pressure on soil thread, friction between hand and soil, friction between soil and base plate, speed of rolling, and geometry of test procedure such as the ratio between hand contact to the diameter of soil thread (Whyte 1982). O’Kelly (2013) states that determination of plastic limit by this method is a reflection of deformability and strength of soil. It is also proposed that the reason for observation of brittle failure in this test is either air entry or cavitation in the soil thread (Haigh et al. 2013; Vardanega and Haigh 2014).

2.4 Criticism of the current standard methods

For having a critical view to current standard methods of determination of soil plasticity, it is required to consider the purpose of geotechnical engineering tests. Selection and application of every soil test should be part of a site characterization program (Germaine and Germaine 2009). Investigation on the physical and hydrological properties of natural soil is the purpose of soil laboratory tests (USACE EM 1110-1-1804 2001). Atterberg limit tests are among index tests. These tests are usually simple, quick, and inexpensive tests that the results provide a general indication of the overall behaviour of soil and used to determine engineering properties of soil utilising more or less empirical correlations (Germaine and Germaine 2009). It should be noted that index tests are generally assigned to characterise the site soil (Germaine and Germaine 2009). It is evident that real scale and in-situ tests provide the most accurate results for this purpose. However, they are too expensive and impractical in many cases. Hence, it is required to develop some laboratory or in-situ index tests for indication of the overall behaviour of site soil. There are many issues and problems related to the current standard methods and apparatus to determine liquid limit and plastic limit of soils that directly and indirectly influence the results of tests. First of all, it is required to consider the nature of these tests. Soil specimen using for these tests are obtained from disturbed soil samples. Although Atterberg tests are among index and classification tests for soils (USACE EM 1110-1-1804 2001), they are determining the engineering parameters of soil, especially remoulded undrained shear strength directly and indirectly. It is evident that disturbed soil samples are not appropriate for the determination of engineering parameters of soils mostly. In the case of liquid and plastic limits tests, it is much worse. Several specimens are prepared from the disturbed samples through the sieving process that makes them fully disturbed soil sample. In this process, the structure of soil will be destroyed and it is almost not possible to correlate the results of Atterberg tests to the site soil. That is why Casagrande (1932) had reported that at the liquid limit determined by liquid limit test, there are some Laurentian clays that are able to carry loads more than 5 tonnes per square foot (≈ 527 kPa) without any excessive settlement. Moreover, in the case of organic soils and especially peat soils, although some remoulded peat may have a plastic range (O'Kelly 2016), it might not be possible to determine the liquid limit by both of fall cone method or Casagrande percussion cup due to effect of fibres (Long and Boylan 2012), and it might also not be possible to roll out a thread of peat soil to the specific diameter of soil thread as specified in BS EN 1377-2 (1990) or ASTM D4318-10e1 (2010) standards (O'Kelly 2016). Hence, it would be reported as non-plastic. In addition, cellular connections and fibres entanglement have a great effect on their shear strength (O'Kelly 2016). By disturbing the samples and especially by sieving, these

connections and fibres entanglement will be destroyed or at least they will be heavily destructed.

It is also required to have a look at the definition of the specimen. “Specimen (from Latin specere, to look at):

1. An example of something from which the character of the whole may be inferred.
2. A part of something taken as representative of the whole.
3. A part or portion of some substance serving as an example of the in question thing for purposes of investigation or scientific study” (Head 2006).

Therefore, it is required to reconsider the definitions and see whether it is still possible to call them specimen or not. Several specimens are prepared from the disturbed samples that their soil particles coarser than 425 microns are removed by sieving. However, there is an unsuitable attempt in BS EN 1377-2 (1990) to correct the moisture content of soil based on the percentage of the particles passed through the 425 µm sieve to compare to the results of liquid limit and plastic limit tests. In this correction, there is no consideration of the effect of the specific surface of soil particles or the soil particle connections and texture. For instance, considering a reinforced soil by net shape tied fibres, as it might be similar to peat soils. By sieving and passing the soil through the 425 µm sieve, the reinforcement will be destroyed and the result of tests will not be accurate at all. In addition, assuming that by considering the third meaning of specimen, these tests are looking for soils’ cohesion that is mainly due to clay particles of soil. Regarding soil particle size ranges (Figure 2-9), it should be limited to soil particles less than 5 micron according to ASTM D422-63 (2002) or less than 2 microns according to BS EN ISO 14688-1:2002+A1 (2013). But soil specimens cover the full range of silt plus some portion of sand too. Author’s efforts to find the main reason for this selection (soil particles with less than 425 microns) in the standards were not successful. But there is a possibility for practical reasons regarding the preparation of soil samples by dry sieving method. Another reason might be due to the limitation of apparatus in the case of containing coarse particles in soil specimens. However, even if coarse particles are considered as neutral materials related to the cohesiveness of soils, it is not a good reason for removing coarse particles without appropriate considerations.

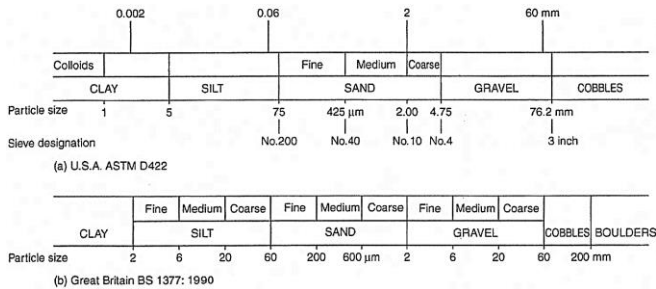


Figure 2-9: Extract from Head (2006). Classification of particle size ranges of soils

Apart from specimen preparation, there are many issues and problems pertinent to the current apparatus of standard methods for determination of soils' liquid limit. First of all, this method is based on an assumption that all soils have a specific small shear strength at their liquid limit (Murthy 2003). However, there are many studies that report a wide range of undrained shear strength at the liquid limit of soils. Prakash (2005) noted this range between 0.5 - 5.6 kPa, however, this might be wider than what are reported in the literature. Hence, assigning a specific shear strength for the liquid limit is not correct for both broad standard methods namely Casagrande (percussion cup) and fall cone methods. However, Casagrande (1958) was one of the most profound geotechnical engineers who suggested this approach for determination of liquid limit of soils.

Various factors and uncertainties affect the results of Casagrande (percussion cup) method. Most of them can be classified into two broad categories namely intrinsic apparatus deficiencies/limitations, and operator's performance/judgement.

Different base material of Casagrande apparatus is being used in various countries. The softer bases absorb more energy, hence less energy will be transferred to the cup of soil. Therefore, higher liquid limit values will be obtained. Haigh (2016) had summarised specification of various Casagrande apparatus in different countries that are presented in Table 2-2. It is also possible to consider this effect for the rubber feet that their role is to isolate the base from the work surface.

Different weights of cup in Casagrande apparatus affects the results. ASTM D2216-10 (2010) lets the range of cup mass between 185 to 215 g which will influence the results obtained by this method.

Two different grooving tool have been used for determination of liquid limit. The ASTM D4318-10e1 (2010) grooving tool which cuts an 8 mm (± 0.1 mm) deep groove and has a surface cutter for controlling the height of two halves of soil in the cup. However, AASHTO T89-10 (2010) curved grooving tool cuts a 10 mm (± 0.1 mm) deep groove with no surface cutter. This will lead to different geometry of soil during the test and therefore different results will be obtained.

In general, calibration condition of Casagrande apparatus greatly affects the results. It is necessary to make sure that the apparatus is fully compatible with the defined test procedure. Most important factors that might affect the results are change in vertical drop height of the cup on base, different rate of drops, using worn grooving tools and cups, and friction in connections of apparatus.

Table 2-2: Specifications for Casagrande liquid limit apparatus in various countries (Haigh 2016)

	Country	Code	Base Hardness*	Equivalent Young's Modulus' (MPa)**	Resilience	Other
Hard Base	USA	ASTM D4318-10e1 (2010)	80-90 D	260-446	77-90 %	
	Canada	CAN/BNQ 2501-090 (2005)	Micarta or hard rubber	-	75-85%	
	Brazil	NBR 6459 (1984)	ebonite	~500	74-92 %	
	Germany	DIN 18122-1 (1997)	>80 D	>260	80-90 %	
	Sweden	SS27119 (1989)	ebonite	~500	-	Superseded by ISO/TS 17892-12 (2004) Fall cone test
	Spain	UNE 103103 (1994)	80-90 D	260-446	75-90 %	
	South Africa		SANS 3001-GR10 (2013)	Hard rubber	-	80-90 %
TMH1 (1986)			85-95 D	340-585	-	
Soft Base	South Korea	KSF-2303 (2000)	83-93 A	11-31	-	
	Japan	JIS A1205 (1999)	83-93 A	11-31	-	Resilience typically 15-40 % (Kazama and Shimobe 1997)
	UK	BS EN 1377-2 (1990)	84-94 IRHD	11-28	20-35 %	Fall cone test preferred
	Australia	AS1289 3.1.1 (2009)	86-94 IRHD	13-28	-	
	India	IS 2720 (Part 5) (1985)	86-90 IRHD	13-18	30-40 %	
Either	New Zealand	NZS 4402 (1986)	79-99 IRHD	8-221	-	
	Switzerland	SN670345a (1989)	Not specified	-	-	
	France	NF P94-051 (1989)	Not specified	-	-	Density 1250-1300 kg/m ³ Compressive strength 180-220 MPa

* The hardness of base material are scaled based on methods of Shore A, Shore D or IRHD

** Approximate conversions of the hardness scales to Young's Modulus are calculated using formula given by Gent (1958) and Hertz and Farinella (1998)

Operator's performance/judgement is more important than intrinsic apparatus deficiency/limitations in obtaining accurate results. Different techniques used by operators during the test procedure is the reason for majority of errors in the liquid limit determination (Sherwood 1970). Different techniques of filling the cup with soil material, method of groove formation, different rate of drops in non-motorized apparatus, and judgment about the closure of groove are among the most important factors that affect the liquid limit values obtained by Casagrande apparatus.

Considering all of the mentioned factors in addition to the huge diversity of soil types reveals that determination of liquid limit of soil is not very repeatable and reliable by Casagrande method. However, if the operator would calibrate/check the apparatus regularly and would follow the standard procedure accurately, reliable results would be obtained. For eliminating many of the uncertainties of percussion cup method, fall cone method was introduced. Many studies have been done to use fall cone method for determination of liquid limit as an alternative method for Casagrande method (i.e. Karlsson (1961; 1977), Lawrence (1980),

Feng (2000), Prakash and Sridharan (2006), Hazell (2008), Lee and Freeman (2009), Guo and Wang (2009), Kayabali (2011a), Emami Azadi and Monfared (2012)) . Although fewer uncertainties were involved in this method, it is based on the inappropriate assumption of a unique shear strength at the liquid limit for all soils that will be discussed in details in “Alternative tests proposed for current standard methods” section. In addition, the performance of operator during the procedure of test still affects the results. Moreover, coarse particles in soil matrix and soil texture affect the penetration of cone into the soil (Germaine and Germaine 2009). Furthermore, fall cone method has some difficulties in reporting non-plastic state for silty soils (Poulsen et al. 2012). Another note is that in the case of soil classification, the results obtained by fall cone method will be compared to the plasticity chart that had been originally established by Casagrande method. All of these show that fall cone method is not also a perfect method for determination of liquid limits of soils.

Results of the plastic limit test by the standard method is mostly depending on the operator. It is very probable to get different results for a soil specimen with different operators. The main reason is due to the many factors involved in the test results and most of them are operator dependent. Figure 2-10 shows bar charts for various plastic limit test results obtained by different operators on four clay soils that show different values obtained by different operators in plastic limit determination using thread rolling method.

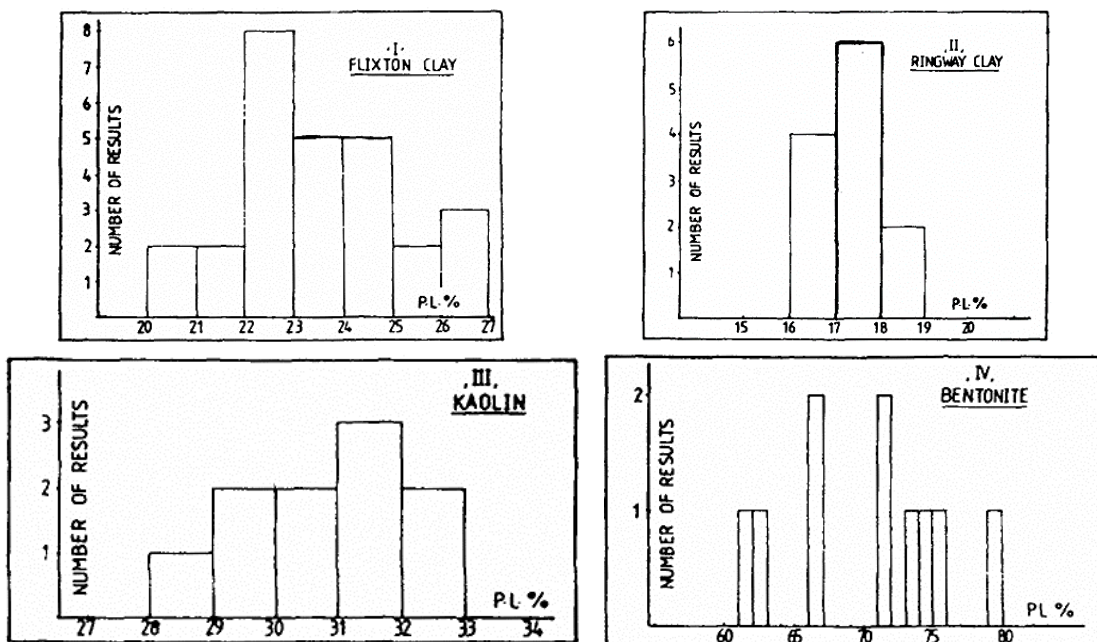


Figure 2-10: Various plastic limit test results obtained by different operators (Medhat and Whyte 1986)

Although there are guidelines for doing this test, many operators will not follow them and even if they want to follow the guideline, the matrix and texture of soil might not let them do that. The amount of pressure differs by rolling the soil thread under fingers at each point. Applying pressure greatly differs proportionate to soil type and plasticity of soil. Rate of rolling differs among operators and is very depend on the soil type. Texture and

form of hand produce different friction between hand and soil. Inaccurate and visual measurement of soil thread diameter when it is crumbling is very common. Ambiguity in definition of crumbling of soil thread is another reason of different values for this test method.

Although the structure and concept of determination of plastic limit by thread rolling method are identical in the main standards, there are a few differences in the procedure of the test in the standards that will affect the results obtained by each method. For instance, requirements or recommendations for glass plates are different in BS and ASTM standards. BS EN 1377-2 (1990) recommends two flat glass plates, one for soil mixing with a size of 10 mm thick and about 500 mm square and one smooth and free from scratches glass plate for rolling threads with a size of about 10 mm thick and 300 mm square. In contrast, ASTM D4318-10e1 (2010) requires a ground glass plate of sufficient size for rolling threads of soils. However, in the past versions of ASTM standards, a ground glass plate was required with at least 30 cm square by 1 cm thick for rolling threads of soil (ASTM D4318-00 2000).

Formation of soil threads is also different in ASTM and BS standards. In hand method of ASTM D4318-10e1 (2010), 1.5 to 2.0 g of plastic soil specimen will be shaped to an ellipsoidal mass and will be rolled between the palm or fingers and the ground-glass plate to form a 3.2 mm (1/8 of an inch) soil thread in a normal rate of rolling recommended between 80 to 90 strokes per minute and within less than 2 minutes. But in BS EN 1377-2 (1990), 20 g of plastic soil specimen will be shaped to a soil ball and will be worked by hand until slight cracks will be visible on its surface. Then it will be divided into two and each portion into four or less equal parts. Then each part will be moulded by first finger and thumb into 6 mm diameter soil thread. After that, the diameter of soil threads will be reduced to 3 mm by rolling between fingers, from finger-tip to the second joint, of one hand and the surface of the glass rolling plate in 5 to 15 cycle of forward and backward movements of fingers depending on the soil type. However, it has been investigated that diameter of soil thread has not a considerable effect on the transition of soil from plastic state to brittle state (Prakash et al. 2009; Barnes 2013; Haigh et al. 2013).

Criteria for reaching to the plastic state of soils have also some differences in ASTM and BS standards. Process of reforming soil from thread to ellipsoidal mass will be continued in ASTM D4318-10e1 (2010) until the thread crumbles and soil cannot be rolled into a 3.2 mm diameter thread anymore. Since crumbling of soil is different for various soil types, the only criterion for stopping the procedure is that the threads can no longer be reformed into an ellipsoidal mass and rolled out again. In contrast, the crumbling condition in BS EN 1377-2 (1990) is seeing both longitudinal and transversal shears in soil thread when it has been rolled to about 3 mm diameter.

Apart from all of these, important questions are:

1. Is the procedure of plastic limit test method appropriate for the determination of the plastic limit state of cohesive soil?
2. Does the condition of soil when the soil thread crumbles at 3 mm diameter replicate the condition of soil in-situ when its moisture content reaches to the plastic limit state?

For answering these questions, it is required to look for the precision, bias, and quality of a test. Precision and bias criteria have been chosen by ASTM to quantify the suitability of a test (Germaine and Germaine 2009). A good test is a test that provides precise and unbiased data. In general, the tests that simulate conditions of the phenomena in the real situation can provide high-quality data. During the thread rolling test, a thread of soil will be under very complex conditions by fast movements of hand to reduce its diameter to 3 mm. During this process, air entry or cavitation causes brittle failure in the soil thread (Haigh et al. 2013; Vardanega and Haigh 2014). This situation will not happen for the soil in-situ. Hence, it is very probable that the measurements provide imprecise and biased data.

After detection of limit states by any method, it is time to determine moisture content. Determination of water content depends on the temperature of the oven. The standard drying temperature varies in different standards (i.e. 105 to 110 °C in BS and 110 ± 5 °C in ASTM for non-organic soils (BS EN 1377-2 1990; ASTM D2216-10 2010)). Adsorbed water is a very viscous liquid. Although adsorbed water plays the most important role in soil plasticity, it is not being determined by this temperature. More than 200 °C is required to remove adsorbed water from a clay particle. It is very important to note that temperature might alter the physical properties of soil material (i.e. by expansion and contraction and change in the specific surface of soil material). Even decomposition might occur in high temperature especially in the case of organic soils. It is suggested to use 60 °C to 65 °C for determination of moisture content of organic soils (BS EN 1377-2 1990). With this temperature, it might not even be possible to measure the hygroscopic moisture of the soil. However, these temperatures will also decompose some contents of organic soils. O'Kelly (2005) investigated level of solid particles oxidation of several soils over different drying temperature range from 60 to 140 °C. He had investigated loss in mass on ignition on the soils at a temperature of 440 °C for a period of 24 hours which the results are presented in Table 2-3. Hence, it is required to consider this topic for further investigations. However, it is not among the objectives of this research.

Table 2-3: Ignition mass loss in some soils (O'Kelly 2005)

Soil type	Classification (USCS)	Liquid limit (%)	Plastic limit (%)	Loss on Ignition (%)
Slit	MH	65	32	3.0
Clay	CH	83	35	5.6
Marl-1	MH	116	95	6.7
Marl-2	MH	131	106	6.9
Shell marl	MH	107	59	10.3
Organic marl	OH	180	130	29.7
Peat-1	Pt	900	310	87.7
Peat-2	Pt	735	485	92.9

2.5 Alternative tests proposed for current standard methods

As discussed, it was widely accepted that soils in their liquid limit state have a small shear strength and it was assumed that the liquid limit values obtained by different standard methods have not very large difference for many soils. In addition, fall cone method had eliminated many uncertainties pertinent to Casagrande percussion cup method. Hence, there have not been many attempts to look for alternative methods for liquid limit determination of soils. However, there are several studies to present unconventional methods for reducing the time of the tests, for obtaining consistent results, and achieving higher degrees of reproducibility. Moreover, there have been some studies to modify the standard methods for determination of liquid limit as some of them had documented in standards, like one point methods of Casagrande and fall cone methods. In contrast, there have been many attempts to find alternative methods for determination of soil plastic limit due to many uncertainties involving in the procedure of plastic limit determination by thread rolling method. Barnes (2013) had comprehensively investigated various studies that had been done to find alternative methods for determination of plastic limit of soils. It is a very good reference on this topic and has been used in this investigation correspondingly. Some of the investigations were concentrated on standardising the thread rolling method by inventing an apparatus to reduce the uncertainties. Some other investigations were concentrated on relating a particular shear strength or a specific parameter of soil to Atterberg limits of soils and determination of that specific shear strength or parameter. Generally, most of the researches utilising this approach have been unsuccessful to become a popular method for determination of Atterberg limits of soils. The key question in this approach is that what value for this specific parameter can be correlated to specific qualitative states of various soil types that many different parameters have effect on the overall behaviour of soil?

Schofield and Wroth (1968) tried to relate critical state model to the undrained shear strength of soils at their liquid and plastic limits. They also relied on the results of the experimental study that had done by Skempton and Northey (1952). That study had been

done on only four different clays and proposed a fixed strengths ratio of 1:100 between the undrained shear strength of soils in their liquid and plastic limits (the results are presented in Figure 2-6). Wroth and Wood (1978) considered the data provided by Youssef et al. (1965) as shown in Figure 2-7 and implemented 1.7 kPa shear strength for liquid limit which was the mean value of the vane shear strength at the liquid limit, and therefore shear strength of 170 kPa became the shear strength of soils at the plastic limit considering the fixed strengths ratio of 1:100. There are some other studies that assumed a different liquid limit shear strength and applied 100-fold strength ratio for plastic limit shear strength. Harison (1988) considered the undrained shear strength of 1.1 kPa for the liquid state of soils and then assigned 110 kPa undrained shear strength for the plastic limit shear strength. In a similar case, Pandian et al. (1993) studied the data provided by Russell and Mickle (1970) and assigned suction of 5-6 kPa for liquid limit, then applied 100-fold strength ratio and suction of 500 kPa assigned to plastic limit of soils. In addition, there are many studies that show various shear strength at liquid and plastic limits. Dennehy (1979) had done several undrained triaxial compression tests on different remoulded clay soils with different water contents that are shown in Figure 2-11.

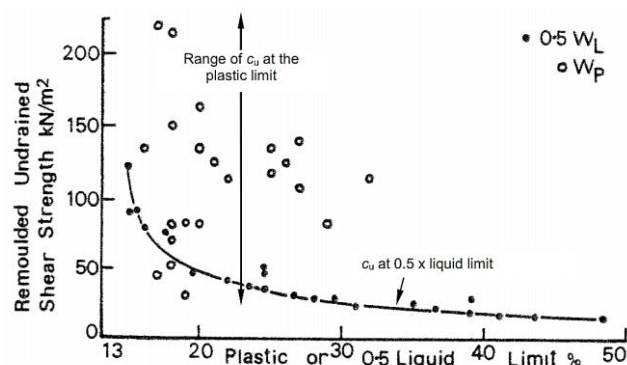


Figure 2-11: Extract from Barnes (2013). Remoulded undrained shear strength versus liquid and plastic limits (Dennehy 1979)

The results showed that when soils were at their plastic limit, the shear strength varied extensively, from about 30 to 220 kPa. The results also did not reveal a common relationship between plastic limit and undrained shear strength.

Black and Lister (1978; 1979) studied some low and high plastic clays that their undrained shear strengths at the plastic limit were varied from about 50 kPa to almost 400 kPa respectively. Their results are shown in Figure 2-12.

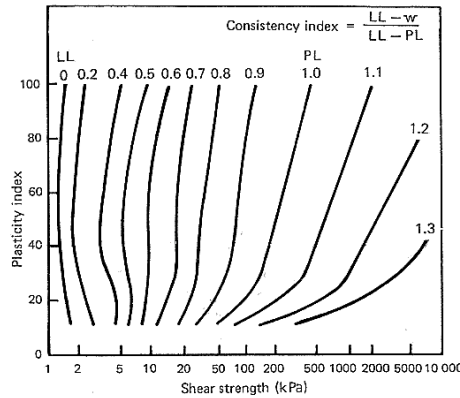


Figure 2-12: Extract from Barnes (2013). Plasticity index versus undrained shear strength (Black and Lister 1979)

Although Stone and Kyambadde (2007) provided data for shear strength at the plastic limit with the range of 65 - 160 kPa, they were persistent about the usage of 100-fold shear strength at the liquid limit for the plastic limit of soil by assuming 1.7 kPa shear strength at the liquid limit. Many other researchers like Karlsson (1961), Dumbleton and West (1970), Whyte (1982), Harison (1990), Marinho and Oliveira (2012), Nagaraj et al. (2012), Vinod et al. (2013) had reported different shear strengths at plastic and liquid limits soils in their researches. Currently, the approach for considering 100 times of shear strength at the liquid limit as an indication for the plastic limit state of soils is recognised as a fallacy and PL_{100} is used for showing the particular water content of soil obtained by this approach.

Nowadays, it is well known that there is no theoretical criterion for considering a fixed strengths ratio (including 1:100) between shear strengths of soils in their liquid and plastic limits. Prakash (2005) mentioned having no rational basis for considering specific shear strength for soils in their liquid and plastic limits and criticised this approach. Nagaraj et al. (2012) also concluded that considering a unique shear strength at liquid limit or plastic limit of soils is not tenable. Hence, assigning a unique value of undrained shear strength for the liquid limits of soils and considering the fixed strengths ratio for determination of undrained shear strength of plastic limit of soil is a fallacy. Utilisation of this approach specifies a particular state of soil and do not determine the plastic limit state of soils that was defined by Atterberg (Haigh et al. 2013). However, many researchers have adopted this approach and many apparatus have been fabricated to determine liquid and plastic limits of soils using this approach. For instance, considering the 80 g fall cone apparatus used in the BS for determination of liquid limit of soils, many researchers considered an 8 kg cone would be appropriate for measuring plastic limit of soils that has 100 times of shear strength of liquid limit. It should be noted that the plastic limit of soils is not a strength-based state and there is only one plastic limit state for each soil. It should also be noted that most of the researchers who tried to justify for a strength-

based plastic limit, used data for well-behaved clays, that show a good plasticity and place above the Casagrande A-line in plasticity chart which are more compatible with the critical state approach of soil mechanics and this concept has not been tested comprehensively for the more problematic soils that are below the A-line in plasticity chart (Barnes 2013).

2.5.1 Determination of soil liquid and plastic limits utilising moisture-tension device

Richards (1948, 1949) developed a moisture tension apparatus for measurement of capillary potential of soils. The apparatus involves several ceramic plate extractors that are able to hold some rubber rings which are mounted in the extractor. The test runs at a specific pressure (usually between 0 to 1 kPa). When the pressure is applied to the extractor, there are several outflow tubes that let the excess water come out of the extractor. Finally, it will reach an equilibrium state and the flow will be ceased (Gadallah 1973).

Rollins and Davidson (1960) tried to study the relationship between soil consistency limits and soil moisture tension. They stated that a specific pressure intensity can be correlated to each soil textural group. They concluded that liquid limit of soils in each soil textural group can be estimated with an acceptable degree of certainty. The results they provided are presented in Table 2-4.

Table 2-4: Soil moisture tension suggested by Rollins and Davidson (1960) for determination of soil liquid limit

Textural Group	Moisture Tension Values		
	Inches of Water	psi	kPa
Clay	6	0.22	1.52
Silty Clay	15	0.54	3.72
Silty Clay Loam	60	2.17	14.96
Silty Loam	60	2.17	14.96

Sultan (1961) worked on Utah soils for his PhD project. He utilised moisture tension method and used textural classification for determination of liquid limit of soils. His results are presented in Table 2-5.

Table 2-5: Soil moisture tension suggested by Sultan (1961) for determination of soil liquid limit

Textural Group	Moisture Tension Values		
	Inches of Water	psi	kPa
Silty Clay Loam	120	4.3	29.65
Loam	110	4	27.58
Sandy Loam	70	2.53	17.44
Sand	40	1.44	9.93
Silty Loam	130	4.3	29.65

Most of the studies that used moisture tension evaluation method were concentrated on the determination of liquid limit of soils. Livneh et al. (1970) worked on the capability of this method for determination of plastic limit and the plasticity index of 20 soils. They categorised these soils in three groups and the classification criteria were particle size distribution and liquid limit values. They provided an equation for the relationship between corresponding suction value and the water content only depending on the soil type. They also proposed another equation to correlate corresponding suction values to the logarithm of the plastic limit and logarithm of the plasticity index of a soil. However, the correlation between suction value and $\log PI$ had a better precision than that with the $\log PL$.

Later, Gadallah (1973) investigated the relationship between consistency limits and the moisture content as obtained by the moisture tension method on 38 soils from Indiana, USA. He tried to investigate the linear relationship between the consistency limits (liquid limit and plastic limit) and the moisture content, obtained by the moisture tension method at various pressure intensities. His research was an attempt for covering the limitations of this method in determination of liquid and plastic limits of soils. Until that time, there was some disagreement relative to the method of preparing soil specimens for the test. He noticed a major effect of preparation method on the results for several soils and suggested that a standardised method of preparation of soil specimen should be used. His study delivered a more consistent determination of liquid limit of soils in comparison to the determination of plastic limit of soils.

2.5.2 Determination of soil liquid and plastic limits utilising extrusion method

Several researchers have tried to utilise extrusion method for determination of liquid and plastic limits of cohesive soils. They have tried to correlate a specific shear strength to the liquid and plastic limit states of soils. This approach was based on the assumption that liquid and plastic limits of saturated remoulded soils outline points on the critical state line that decrease in water content from liquid limit to plastic limit corresponds to an arbitrary proportional increase of strength (Schofield and Wroth 1968).

Extrusion method is a technique in which materials that can be deformed (i.e. food paste, melted plastics, metals, fine-grained soil, etc.) are induced to flow through an aperture under pressure by an extruder or a rammer (Medhat and Whyte 1986). Detailed mechanism of this method is explained in “Mechanism of Extrusion Method” section in chapter 3.

Timar (1974) was the first one who utilised direct extrusion method for determination of liquid and plastic limits of soil. Later, Whyte (1982) utilised reverse extrusion method to

find a connection between moisture content of soils in plastic and liquid states with specific shear strengths. A schematic illustration of direct and reverse extrusion processes is shown in Figure 2-13.

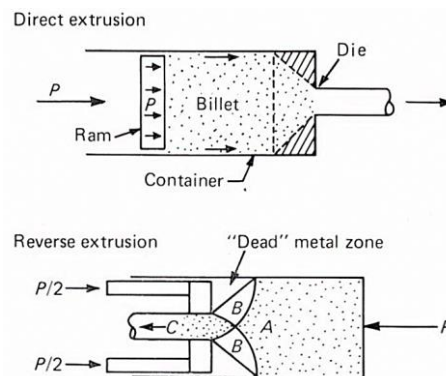


Figure 2-13: Extract from Whyte (1982). Direct and reverse extrusion processes

As it is mentioned in Figure 2-13 ("Dead" metal zone), Whyte had used pattern of metal extrusion in his investigations. Whyte (1982) considered $c_u = 1.6$ kPa and $c_u = 110$ kPa for liquid limit and plastic limit respectively based on the data he obtained from various researchers (Table 2-6 and Table 2-7). In case of shear strength at the plastic limit of soils he has mentioned that $c_u = 110$ kPa also is a representative of consistency of soils around mid-way of "stiff" range. However, he had not stated that based on what data and classification he had concluded this outcome.

Table 2-6: Shear strength at liquid limit (Whyte 1982)

Base Material	c_u (kPa)	Source
Ebonite	2.5	Seed et al. (1964)
Micarta	2 – 3	Casagrande (1958)
	1.1 – 2.3	Norman (1958)
	1.3 – 2.4	Youssef et al. (1965)
	0.5 – 4.0	Karlsson (1977)
BS Rubber	0.8 – 1.6	Norman (1958)
	0.7 – 1.45	Skempton and Northey (1952)
	1 – 3	Skopek and Ter-Stepanian (1975)

Table 2-7: Shear strength at plastic limit (Whyte 1982)

c_u (kPa)	Average c_u (kPa)	Source
85 – 125	110	Skempton and Northey (1952)
30 – 320	115 (arithmetic) 104 (geometric)	Dennehy (1979)
20 – 220	110	Arrowsmith (1979b)
25 – 280	130	Arrowsmith (1979a)
170 assumed	-	Wroth (1979)

Whyte (1982) proposed that this technique is a reliable method for determination of plastic limits of soils due to several advantages that it has. Requirement of a small portion of a soil sample. Moisture content of soil specimen can be changed gradually and be extruded as long as it is required. Based on the design of extrusion apparatus, it is

possible to have a simple, robust and inexpensive apparatus for the test. The results obtained by this method has a better reliability and reproducibility in comparison to other alternative strength tests (i.e. laboratory vanes, cones, penetrometers and unconfined shear test). The results obtained by this method are also correlated to the results of undrained triaxial strength tests. It is a relatively quick and easy test that provides much more reliable results in comparison to conventional methods. However, Whyte (1982) investigated this method on only one soil type to determine the undrained shear strengths at the liquid and plastic limits of the soil. In addition, based on the chosen shear strengths at liquid and plastic limits, he had considered the ratio of shear strength at the plastic limit to the liquid limit about 70 (68.75 accurately) which is different with the ratio considered by other researchers who have followed strength based approach. He provided an empirical relationship between the extrusion pressure, remoulded shear strength, container and die orifice diameters for only one low plastic clay ($LL = 32.5\%$, $PL = 16.5\%$) and he chose the ratio of 40 for container and die orifice surface areas (38 mm and 6 mm diameters respectively).

Kayabali and Tufenkci (2007) tried to estimate both liquid and plastic limits of soils utilising reverse extrusion method more comprehensively for the first time. The principles of Whyte's paper (1982) were applied on twenty soil samples for determination of their plastic limits. However, they tried to correlate the extrusion pressure to the results obtained by current standard methods. Their apparatus had also a ratio of 40 in cross-sectional area of the container to the cross-sectional area of the orifice of the die as it was chosen by Whyte (1982). The apparatus they used is shown in Figure 2-14.



Figure 2-14: Extract from Kayabali and Tufenkci (2010a). Kayabali and Tufenkci's apparatus components

They used a die orifice of 6 mm diameter and reported 30 kPa and 2250 kPa as the extrusion pressures for liquid and plastic limits respectively. They also mentioned that different die orifice and container diameters should lead to various extrusion pressures. They continued their research and investigated the repeatability and dependency of test to its operator (Kayabali and Tufenkci 2010a). By conducting reverse extrusion tests in addition to Casagrande percussion cup and thread rolling test methods, they reported

new extrusion pressures for liquid and plastic limits, 35 kPa and 3000 kPa respectively. A sample plot of their investigation is shown in Figure 2-15. Their research showed that this method is not dependent on operator and they also noted its potential capability for determination of shrinkage limit of soils.

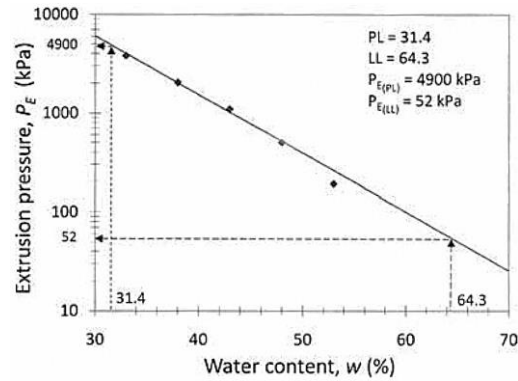


Figure 2-15: Sample plot for obtaining the extrusion pressures corresponding to the Atterberg limit values from standard methods (Kayabali and Tufenkci 2010a)

Kayabali (2012b) noted degree of scattering of extrusion pressures around the mean value for their previous researches and related it to different operators for doing conventional liquid and plastic limit tests. He tried to check and confirm the estimation of liquid limit and plastic limit based on the reverse extrusion pressures (Kayabali 2012b). He increased the number of soil samples and number of tests performed on each sample to more than 4000 consistency limits and reverse extrusion tests. He also studied the results obtained by one particular operator that performed the consistency tests. He concluded that 20 kPa, 2000 kPa, and 12000 kPa are respectively extrusion pressures values for the liquid limit, plastic limit and shrinkage limit of soils. It should be noted that the extrusion pressure values of his last two studies are very different (decrease of 42.86% and 33.33% for liquid and plastic limits respectively). He mentioned that, although back calculations of liquid limit determination using reverse extrusion method had a significant agreement with liquid limits obtained by Casagrande method, a very good correlation did not obtain for plastic and shrinkage limits.

Kayabali et al. (2015b) reassessed the capability of his method for determination of liquid and plastic limits by investigating 70 fine-grained soils with a relatively large range of plasticity with liquid limits ranging from 29 to 105%. They criticised the previous approaches of utilisation of extrusion method for prediction of Atterberg limits and expressed that the representative extrusion pressure was not encouraging in this regard as it was considered in the past. They suggested a multiple regression analysis of the results obtained by the Kayabali and Tufenkci's apparatus and proposed reverse extrusion coefficients for correlating the results obtained by their method to the results obtained by standard test methods of fall cone and rolling-device for liquid and plastic

limits respectively. In the proposed method, more than five extrusion tests were done on each soil sample with various water contents. The plot of extrusion pressures against water content could provide the reverse extrusion coefficients (the coefficient a for y-intercept and the coefficient b for slope of the curve for evaluation of specific values of extrusion pressures at liquid and plastic limits). In this method, they had tried to determine the liquid or plastic limits as a function of reverse extrusion coefficients. The data shapes a semi-logarithmic graph of extrusion pressures against water content which a sample is shown in Figure 2-16.

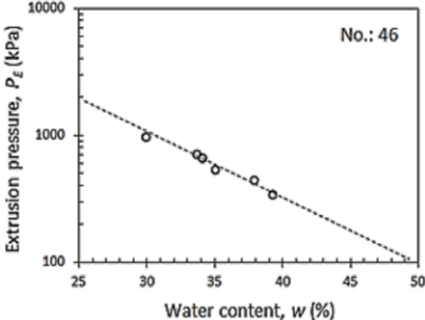


Figure 2-16: Example plot for the extrusion pressures versus water content (Kayabali et al. 2015b)

By considering a multiple regression between the coefficients of reverse extrusion and the results obtained by conventional methods, they provided the following empirical relationship for determination of liquid and plastic limits respectively:

$$LL = 0.04(a^{3.3})1.135^b \tag{2-1}$$

$$PL = 0.04(a^{2.33}) b^{0.98} \tag{2-2}$$

where a and b are the coefficients of y-intercept and slope of the plot of extrusion pressures against water content respectively. Kayabali et al. (2015b) had claimed the results obtained by this method for determination of liquid and plastic limit have a great degree of success with less than $\pm 10\%$ error for both limit states.

Kayabali et al. (2016) developed a new extrusion method for determination of liquid and plastic limits of soils (mud press method). They had used a direct extrusion device with multi-hole mould that the components of the device are shown in Figure 2-17.



Figure 2-17: Mud press device (Kayabali et al. 2016)

They had not provided any reason for choosing a multi-hole container, the size of holes, and the number of holes. They stated that all of them were chosen arbitrarily. They investigated 275 soil samples covering the liquid limits with the range of 28-166% and used a regression analysis similar to their previous investigation for prediction of liquid and plastic limits by following equations:

$$LL = 97.6 + 378 \log a + 3.29^{1/b} \quad (2-3)$$

$$PL = 3.27 + 13.2a - 453b \quad (2-4)$$

where a and b are the y-intercept and the inverse slope of the plotted MPM force against water content data respectively. It was claimed that Equations (2-3) and (2-4) had predicted the liquid and plastic limits of the research soils with 5.6% and 6.4% mean of the errors and a standard deviation of 4.1% and 4.4% respectively. They attributed the lower accuracy of plastic limit prediction in comparison to the liquid limit prediction to the very speculative nature of the determination of plastic limit by thread rolling method. For each soil sample, they prepared more than 5 specimens within the plastic range of soil from near liquid limit to less water contents by adding dry soils to the previous specimen. Although they had mentioned the requirement of maturing the soil sample for at least 16 hours based on ASTM D4318-05 (2005), they had not followed the procedure and prepared the specimens from wet side to dry side by adding dry soil to the soil paste. It should also be noted that the procedure they had proposed is a prediction of liquid and plastic limits obtained by conventional methods and not a direct determination of these limits.

2.5.3 Gay and Kaiser thread rolling method

Gay and Kaiser (1973) devised an apparatus including three vertically rollers rotating by an electric motor. It was required to form a soil thread with about 8 mm diameter and 50 mm long when the soil was in a state of near plastic limit and mounted into the top of the rollers. The thread of soil was rolled out at the bottom of the apparatus with a diameter of about 3.2 mm but with different length based on the water content of the soil thread. They stated that the threads which are broken with 38.1 mm length are at the plastic limit state.

The test proposed by Gay and Kaiser is a tension test that breaks the threads based on soil tenacity which is the potential of soil in holding together the soil particles. However, it does not show the transition from ductile to brittle state of soil (Barnes 2013). It is very unlikely that this device could roll out a thread of soil with the length they specified at its plastic limit which becomes brittle. In addition, the soil they used were usually tough soils

that are able to hold together and are usually above A-Line in plasticity chart and they did not test soils below this line (Barnes 2013).

2.5.4 Bobrowski and Griekspoor thread rolling method

Bobrowski and Griekspoor (1992) devised a thread rolling device that included two flat plates covered with some papers that do not add fibres to the soil. In this apparatus, a soil thread diameter can be reduced to 1/8 inch (about 3.2 mm) by pushing the top plate and moving it forward and backward at the same time. The main purpose of this device was to simulate the same procedure of standard thread rolling method that was done by hand and produce soil threads with exact 1/8 inch diameter. The other criteria for detecting soils in plastic limit were generally like the standard method. They emphasised the usage of those particular papers that rolled the soil threads and not sliding on it, which would lead to a faster drying process. Their apparatus is shown in Figure 2-18.

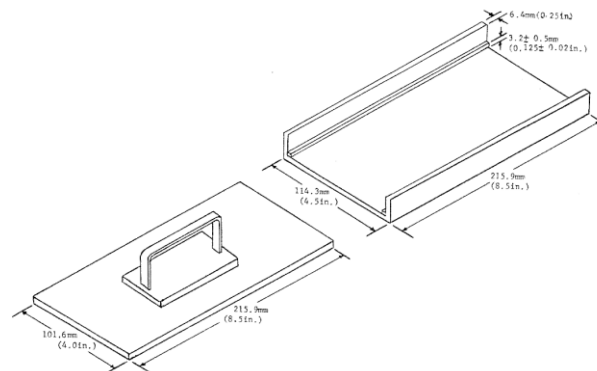


Figure 2-18: Extract from Barnes (2013). Bobrowski Jr and Griekspoor's apparatus components

They only investigated some soils with a plastic limit range of 12 – 20% and the results obtained by their device were almost entirely less than the thread rolling method by hand (Barnes 2013). Another problem is that it is not possible to watch thread of soil when it is getting thinner unless by removing top plate as it is usually would be checked when the diameter has reached to 1/8 inch. In addition, the roughness of the papers that cover the plates provides resistance to soil thread elongation. This resistance can also be seen due to the flatness of the top and bottom plates. The papers accelerate the drying process at the circumference of soil thread which will lead to a crust that is much dryer than the inside of soil thread and will lead to a non-uniform material (Barnes 2013). Although this will also happen in the case of hand method, this process is very dependent to the degree of roughness of papers.

Rashid et al. (2008) studied different modified methods of soil plastic determination including this method. Their research covered soils with plastic limits of more than 20%. They also reported that values obtained by Bobrowski and Griekspoor thread rolling

method generally provide less plastic limit values than conventional thread rolling method by hand.

2.5.5 Temyingyong et al. thread rolling method

Temyingyong et al. (2002) modified Bobrowski and Griekspoor thread rolling method and used a mechanised top plate that was able to move forward and backward with adjustable rates and with different pressure caused by various weights exerted on the top plate. Their apparatus is shown in Figure 2-19.

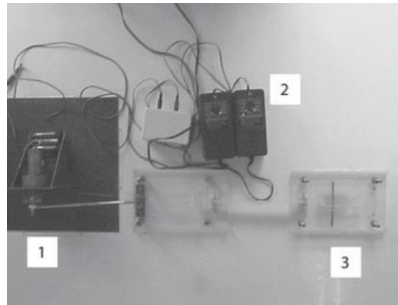


Figure 2-19: Temyingyong et al. apparatus components (Temyingyong et al. 2002)

They also investigated various initial dimensions of soil thread and different surface roughness. They pointed out two main factors affecting the plastic limit values of soils: initial size of the soil sample and the type of soil classified by plasticity. They concluded that other mechanical factors such as friction, speed, and pressure have little effect on the plastic limit value of soils based on statistical analysis of their research which seems incorrect for different soil types. For instance, friction has a direct relation to the roughness of two surfaces and the more friction, the more resistance to soil thread elongation. In addition, the rougher the surfaces, the quicker drying process at the circumference of soil thread which will lead to non-uniform soil specimen during the test. Moreover, the amount of speed and pressure have a direct effect on drying process and crumbling propagation of soil thread in the test that different standards have expressed specific conditions for these factors (BS EN 1377-2 1990; ASTM D4318-10e1 2010). It should be noted that their research did not study a large variety of soil types with various soil plastic limits. The plastic range before and after controlling influence factors were 22 - 34% and 28 - 30% respectively.

2.5.6 Barnes thread rolling method

Barnes (2013) devised a new apparatus replicating the thread rolling method. In this apparatus, a thread of soil is input and will be extruded with a less diameter by rolling between two plates. It is tried to decrease the operator role in the reduction of soil thread diameter. His apparatus is shown in Figure 2-20.

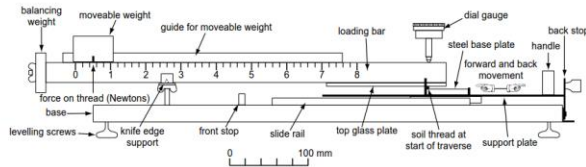
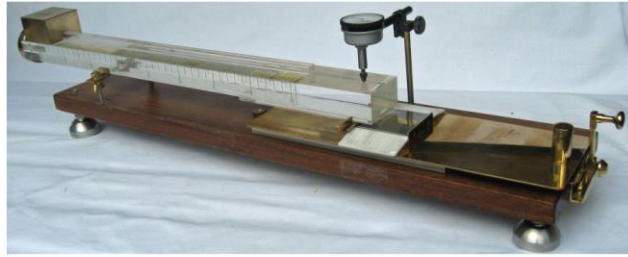


Figure 2-20: Barnes thread rolling device (Barnes 2013)

Nominal stresses and diametrical strains are obtained during rolling the soil thread. Then stress – strain plots are drawn and workability or toughness of the plastic soil are determined as the work per unit volume of soil to eliminate the personal judgement of the crumbling condition that happens to soil threads at the plastic limit. In this research, a wide range of moisture content from near the sticky limit to the brittle state had been studied. He had tried to provide a definition of plastic limit for soils based on workability or toughness of soils for determination of ductile-brittle transition of soil at the plastic limit. The ductile-brittle transition of soil can be observed in a diagram of nominal stress against diameter of soil thread in Barnes apparatus as it is shown in Figure 2-21 for soil threads in tests 21 and 22 which had had brittle and ductile manners respectively.

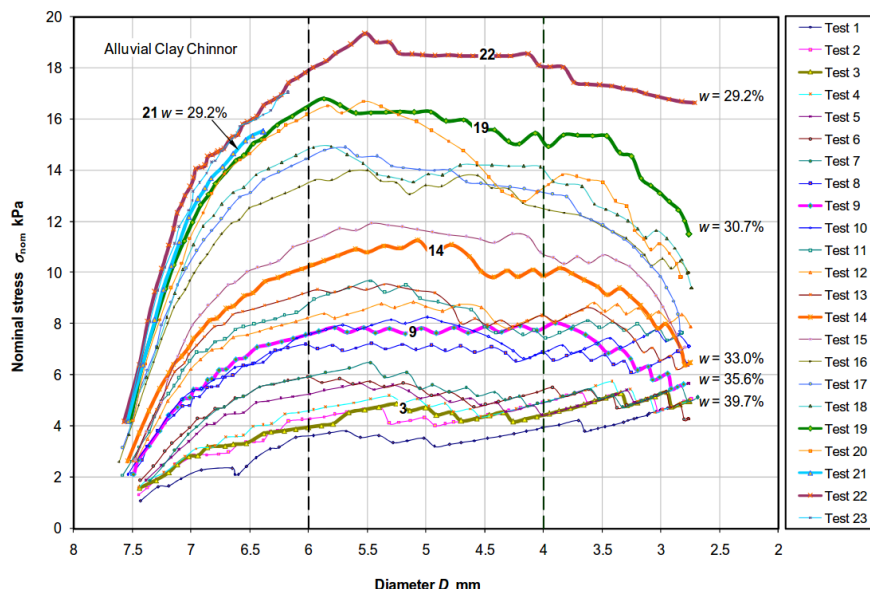


Figure 2-21: A graph sample of nominal stress against diameter of soil thread in Barnes apparatus (Barnes 2013)

In addition, Barnes (2013) had claimed: “The ductile-brittle transition has been found to be sharp and distinct, usually within a fraction of a percentage point of water content for

all of the soils tested”. However, the behaviour of soil do not change immediately by varying water content as Terzaghi et al. (1996) state: “the transition from one state to another does not occur abruptly as soon as some critical water content is reached. It occurs gradually over a fairly large range in the value of the water content. For this reason, every attempt to establish criteria for the boundaries between the limits of consistency involves some arbitrary elements”. Moreover, the results obtained by Barnes apparatus were more sensitive to the presence of coarser grains than those from the hand rolling method. Hence, usage of samples that were prepared by natural soil preparation method had some difficulties. The plastic limit results obtained by Barnes apparatus were generally near the results from usual thread rolling method.

2.5.7 Plastic limit determination by fall cone method

As it was discussed earlier, many researchers assumed a small shear strength for the liquid limit, then applied a strength ratio factor (usually 100) to consider a specific shear strength for the plastic limit state of soils. Therefore, they tried to measure that particular shear strength by different apparatus, especially cone penetration method. However, this method measures remoulded shear strength of soils and it is not able to measure undrained cohesion which is related to the plasticity of soil (Prakash and Sridharan 2006). Sridharan and Prakash (1999) had explained two components of the undrained strength of a soil (undrained cohesion and undrained frictional resistance). The undrained cohesion component is due to viscous diffuse double layer water around soil particles and undrained frictional resistance is because of soil fabric which is the result of the net interparticle attractive forces (Prakash and Sridharan 2006). Fall cone method determines the undrained strength mostly due to undrained friction which cannot be representative of plasticity property of soil (Prakash and Sridharan 2006). Hence, the results can be obtained even for non-plastic soils that make this method unreliable.

Wood and Wroth (1978) used Hansbo's equation (1957) for determining undrained shear strength utilising fall cone method:

$$c_u = k \frac{W}{d^2} \quad (2-5)$$

where k is cone factor, W is the weight of the cone, and d is the depth of cone penetration. The cone factor depends on cone surface roughness and cone angle (Houlsby 1982; Wood 1985). It should be noted that soil type would also have an effect on this factor because of different friction coefficients between surfaces of the cone and various soil particles. Hence, soils with coarser and rougher particles (i.e. silts and sands) would resist more against the cone penetration (Barnes 2013).

According to this equation and assuming a specific shear strength for liquid and plastic limits of soils, it is possible to use fall cone method for determination of plastic boundaries of soils by considering specific cones and cone penetration depth in a particular time duration. However, it was discussed that this is a wrong approach. Nevertheless, this approach has been used in different countries for decades and many researchers adopted various cones with different geometry, weights and penetration depths in soils filled in different containers to determine the plastic limit of soils with fall cone method. Medhat and Whyte (1986) used a cone with 30° cone angle with constant penetration rate to derive cone force versus penetration depth plots. They considered 110 kPa shear strength at the plastic limit of soils with a cone penetration of 10 mm. They used a soil container like liquid limit determination by fall cone method and filled the container with statically compacted soil. They showed that for a specific clay (Flixton Clay), 10 mm of cone penetration could indicate a water content of more than the plastic limit. Harison (1988) noticed the non-linearity of liquidity index versus shear strength plot based on the data provided by Skempton and Northey (1952) and proposed two straight lines that were broke at liquidity index with 0.77 value as it is shown in Figure 2-22. He plotted water content against log cone penetration graph and considered 2 mm of BS cone test for the plastic state of soils. He expressed that there is a linear relationship between cone penetration of 14 mm and 2 mm and proposed that it is possible to extrapolate 2 mm of cone penetration by having a cone penetration of 5, 10, and 14 mm. However, Feng (2000) expressed that utilising this approach will lead to an under-estimate determination of plastic limits. In addition, Harison (1990) had stated that fall cone method has its own limitations and values below 5 mm cone penetration are not reliable.

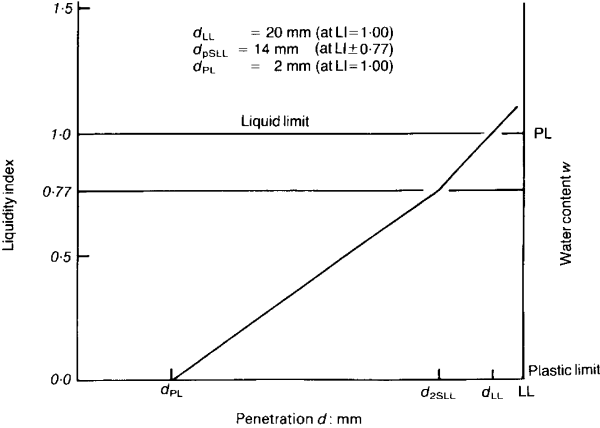


Figure 2-22: Liquidity index and depth of cone penetration (Harison 1988)

Stone and Phan (1995) used a cone penetrometer with a 30° cone angle for determination of penetration force required for steady monotonic penetration into clay samples and suggested usage of their instrument for determination of water content of

a soil with a strength 100 times that at the liquid limit. They studied a kaolin and a brown clay. Although the data they provided showed close shear strengths at the liquid limit of the soils, the shear strengths at plastic limits were very different (50 kPa and 210 kPa for the kaolin and the brown clay respectively as shown in Figure 2-23).

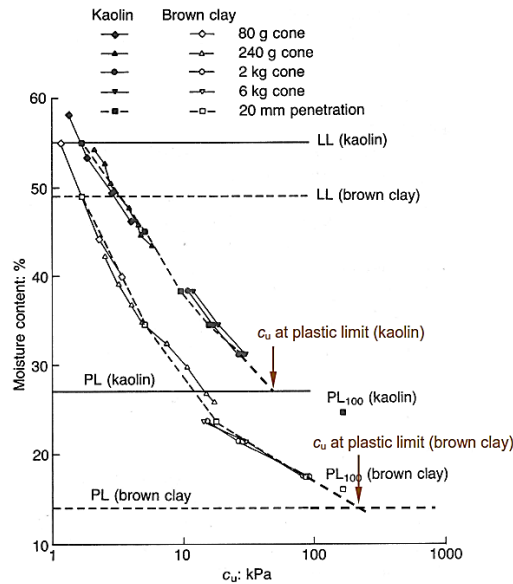


Figure 2-23: Extract from Barnes (2013). Shear strength at the plastic limit (From Stone and Phan 1995)

Feng (2000) proposed plotting the water content against cone penetration on a log-log scale plot. According to the values obtained by his apparatus, he provided an equation based on 2 mm penetration of cone in the soil:

$$w_p = c(2)^m \quad (2-6)$$

where c is the water content at penetration depth (d) of 1 mm, and m is the slope of the flow curve as they are shown in Figure 2-24. However, it was still necessary to extrapolate the 2 mm penetration of cone. The results obtained by his apparatus were within the range of 0.8 to 1.2 of the results obtained by standard hand thread rolling method.

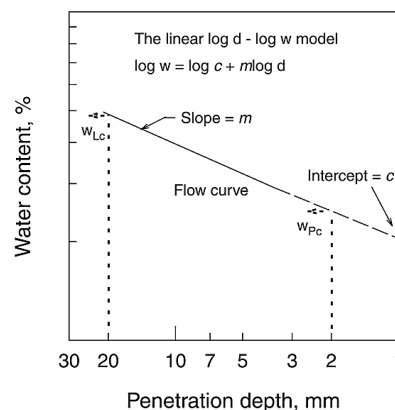


Figure 2-24: An example of linear logarithmic water content against logarithmic penetration depth model (Feng 2001)

Feng (2001; 2004) provided another equation for the one-point method for determining the plastic limit of soils by the fall cone method:

$$w_p = c(d)^m \tag{2-7}$$

where all parameters had defined before.

Feng (2004) also proposed another equation with dividing equation (2-6) by equation (2-7) and considering average value for m (0.265 which was derived from m values of 0.174 to 0.395):

$$w_p = w \left(\frac{2}{d} \right)^{0.265} \tag{2-8}$$

where w is the water content, and d is the cone penetration.

Koumoto and Houlsby (2001) used a cone with a mass of 60 g and cone angle of 60° which would penetrate 10 mm for determination of plastic limit of soil. They expressed that plot of shear strength versus soil moisture content is not linear and concluded that extrapolating from log-log scale plot of cone penetration versus soil moisture content is more appropriate.

Sharma and Bora (2003) used a cone with an angle of 30° and weight of 3.92 N that 4.4 mm penetration of the cone would indicate of the plastic limit of soil. Although they reported a good correlation between the results obtained by their apparatus and the results obtained by standard thread rolling method, the plastic limits determined by their apparatus were usually less than the values obtained by thread rolling method as it is demonstrated in Figure 2-25.

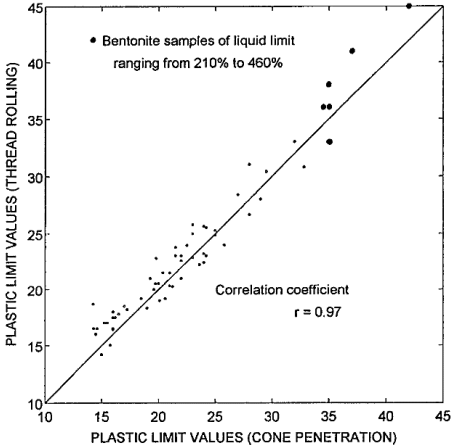


Figure 2-25: Comparison of plastic limit values obtained from thread rolling method and cone penetration (Sharma and Bora 2003)

Muntohar and Hashim (2005) reported the non-linear relationship between log cone penetration depth and liquidity index of soils. They considered 2.2 mm of cone penetration for determination of plastic limit of soils and they used a non-linear fit curve for extrapolation of this cone penetration. They provided 28 points in their graph

(Figure 2-26) and concluded a “very satisfy correlation” between the results obtained by their device and the results obtained by standard thread rolling method (coefficient of determination, $R^2 = 0.852$).

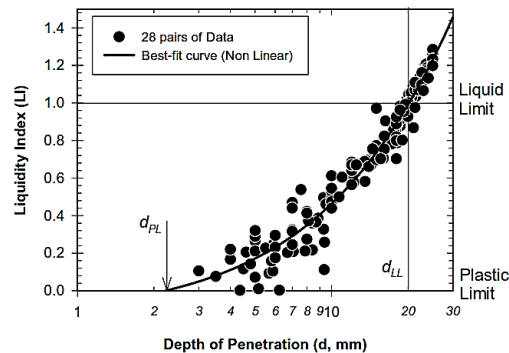


Figure 2-26: Correlation of cone penetration and liquidity index (Muntohar and Hashim 2005)

Dolinar and Trauner (2005) demonstrated that there is a non-linear relationship between cone penetration depth and soil moisture content. However, they assumed a linear relationship between these values in log-log scale plot. They considered a 100-fold strength ratio between plastic and liquid limit shear strengths and used the Feng’s (2000) equation for relating soil water content to cone penetration depth. The results of their research were depended on size and amount of the clay minerals, and specific surface of clay soils.

Stone and Kyambadde (2007) suggested a quasi-static penetration test using a soil mini-penetrometer (SMP) for direct measurement of undrained shear strength of fine-grained soils and introduced a new plastic limit parameter (PL_{100}) that enables to determine water content of a soil with an undrained strength 100 times that at the liquid limit of that soil. Lee and Freeman (2009) proposed a dual-weight fall cone device that was able to change the weight of a 30° cone rapidly. They assumed the 100-fold strength ratio between plastic limit and liquid limits of soils and considered 2 mm penetration of cone for determination of the plastic limit state of soils. The results obtained by their device were usually less than standard thread rolling method.

Sivakumar et al. (2009) altered the BS fall cone apparatus and applied a 54 N fast-static load on the cone. They investigated 16 different clays and considered 20 mm penetration of cone in soils in 15 seconds as the indication of the plastic limit state of soils. They also concluded that many researchers had suggested 8 kg for providing required force for 20 mm penetration of cone in soil was far more than what is required.

Sivakumar et al. (2015) devised a fall cone apparatus using an energy-based measure for penetrating a 0.727 kg cone of 30° apex angle falling from a 200 mm height into the soil specimen. They considered 20 mm of cone penetration as the indication of plastic

strength limit state of soil as it is shown in Figure 2-27. They investigated ten intermediate to very high plasticity clays and concluded a good agreement with the results of measured Casagrande plastic limits. They also investigated the usage of 8 kg contacting cone that was suggested by many researchers and concluded that the results obtained by this cone were often lower than the Casagrande plastic limits.

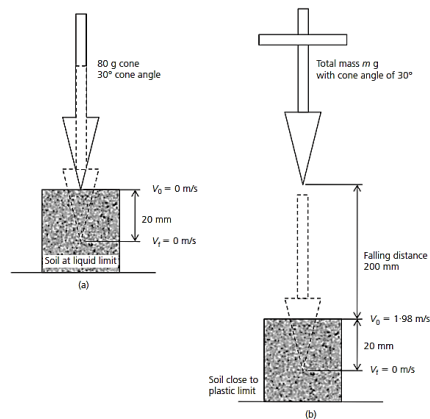


Figure 2-27: Fall cone method description at (a) liquid limit and (b) plastic limit (Sivakumar et al. 2015)

Barnes (2013) had criticised utilisation of fall cone method for determination of plastic limit of soils. He had provided several reasons in this regard:

1. It is incorrect to assume a unique undrained shear strength for soils at their plastic limit state.
2. It is incorrect to assume the 100-fold ratio of strength at the plastic limit in comparison to the liquid limit.
3. It is very hard to prepare a homogeneous saturated specimen for determination of plastic limit by fall cone method.
4. The designation of the cone has many uncertain effects on the results of the test.
5. The relationship between the depth of cone penetration and water content is uncertain.
6. It is necessary to extrapolate data from soil moisture content versus cone penetration depth.
7. Tests have been conducted on well-behaved soils above A-Line in plasticity chart.

2.6 Criticism of alternative proposed tests

There have been many studies to eliminate the errors and uncertainties pertinent to the regular standard methods of Atterberg limits determination. In general, most of the alternative test methods for determination of Atterberg limits can be classified into two broad categories with different approaches.

The first approach has been a replication of conventional standard methods by eliminating uncertainties and personal judgements pertinent to the standard methods.

Following this approach, researchers have tried to provide a correlation between the results of their apparatus to the results of standard methods for determination of Atterberg limits. The accuracy and reliability of conventional methods are the prerequisites for accuracy and reliability of the alternative test methods with this approach. Regarding the intrinsic problems of conventional methods discussed in the past, this is not a correct approach for determination of Atterberg limits of cohesive soils. The second approach is a determination of a specific parameter of soil (e.g. shear strength) in a test method and considering a particular value of that parameter as the indication of Atterberg limit states. This approach has led to different new definitions for plastic or liquid limits which are different with what is described by Atterberg. In addition, in the case of the plastic limit determination, it is unexpected to find a proper correlation between the results obtained by this approach which is a measurement of only one parameter and the results of thread rolling method which is related to many variable factors as explained before. Moreover, it should be asked that, what test can provide reliable values for determination of remoulded shear strength at specific states of soil to be a benchmark for results obtained by alternative methods? It should be noted that the results obtained by different methods are a function of different parameters such as test method assumptions, operator's performance, strain rate, volume of soil, stress history, boundary conditions, and etc. For instance, inconsistent results will be obtained in the determination of shear strength of soils at plastic limit due to the various stiffness of soils, difficulty in mixing the soil paste, partially saturated condition of specimens, and different compaction of soil specimens. Although the values obtained by this approach can be useful in providing more information about different states of soil with varying water content, they generally cannot be representative of the overall behaviour of soil which involves many different parameters.

Almost all of the proposed alternative methods for determination of liquid and plastic limits have followed these two main approaches and they have not been successful or popular in most cases. As long as these two main approaches are followed for determination of Atterberg limits, the same inconsistent results will be obtained by different alternative test methods and the desired goal for determination of accurate soil plasticity will not be achieved.

2.7 Discussion

Atterberg limit tests are among soil index tests that are meant to evaluate the consistency of cohesive soils which is the most important soil aggregate properties of these soil types. Soil consistency is a term that expresses the level of firmness of cohesive soils which is a qualitative phenomenon. It depends on the behaviour of soil while its moisture content

varies. It is very important to note that consistency determination of cohesive soils is the main purpose of Atterberg limit tests and the results of Atterberg limit tests are indications for the transitional behaviour of cohesive soil from one state to another state. In other words, determination of qualitative terms of soil consistency, as it is denoted in Figure 2-2, is the main subject of Atterberg limit tests. ASTM D653-14 (2014) generally defines “consistency” as: “the relative ease with which a soil can be deformed”. For clarification of the meaning of “consistency”, it is useful to have a look at its definition in Oxford Dictionary of English (Stevenson 2010): “Consistent behaviour” and provides more information: “the quality of achieving a level of performance which does not vary greatly in quality over time” (author’s underlining). The meaning of “consistent” is: “acting or done in the same way over time, especially so as to be fair or accurate” (Stevenson 2010) (author’s underlining).

The underlined parts of the meanings are the most important points that should be considered. These parts clarify that the behaviour of a phenomenon should be monitored over a period of time to investigate its consistency. According to this, it is possible to have a comparison between the behaviour of different materials in a specific system and period of time, and judge about the level of performance of the materials.

It should be noted that all cohesive soils should have the same consistency at their liquid or plastic limits. However, their water content might be different at these limit states. In other words, the overall behaviour of different soils with different water contents should be similar to each other at limit states. In this regard, the transitional behaviour of soil due to change of water content should be investigated and a specific classification should be applied for all soils.

Considering the main definitions of liquid limit and plastic limit in the determination of soil plasticity is of great importance. Liquid limit in cohesive soils is defined as “the water content, in percent, of a soil at the arbitrarily defined boundary representing the transition from the semi-liquid to plastic states” in ASTM D653-14 (2014) or defined as “the empirically established moisture content at which a soil passes from the liquid state to the plastic state” in BS EN 1377-2 (1990) (author’s underlining). Plastic limit in cohesive soils is defined as “the water content, in percent, of a soil at the boundary representing the transition from the plastic to semi-solid states” in ASTM D653-14 (2014) or defined as “the empirically established moisture content at which a soil becomes too dry to be plastic” in BS EN 1377-2 (1990). As it is evident, the words “arbitrarily” and “empirically” in definitions is because of quantifying a qualitative phenomenon and incapability of current standard methods for accurate and consistent determination of this state in various soil types. This adverb is also required to be put in the plastic limit definition until a precise method has not established.

Another point is that these definitions are not accurate enough to explain the exact state of soils at their limit states. Unfortunately, these standards do not explain different features of soil in different states properly. For instance, considering the liquid limit definitions, ASTM D653-14 (2014) and BS EN 1377-2 (1990) do not have any definition for “semi-liquid” state and “liquid state” respectively. ASTM D653-14 (2014) does not express the meaning of “semi-solid state” either. In addition, do these standards pointing the same states with different terms in liquid limit definitions? Furthermore, although BS EN 1377-2 (1990) does not have any definition for “plastic state”, ASTM D653-14 (2014) defines “plastic soil” as “a soil that exhibits plasticity”, and “a soil which has a range of water content over which it exhibits plasticity and which will retain its shape on drying” (author’s underlining). Hence, it is necessary to know the meaning of plasticity to recognise plastic soils from non-plastic soils, but let’s first discuss the end part of the definition: “...which will retain its shape on drying”. This should not be true for cohesive soils. By drying a soil in a plastic state, the shape of most soils will change. For instance, one way of determination of shrinkage limit is the measurement of volume change from wet side to dry side which the shape of soil specimen clearly changes. Since the shrinkage limit is less than the plastic state of the soil, the shape of plastic soil will certainly change by drying. In addition, the shape of solid particles and structure of many organic soils will change by drying which will lead to shape change of soil specimen. Plasticity is also defined as: “the property of a soil or rock which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change”, and “property of a material to continue to deform indefinitely while sustaining a constant stress” (ASTM D653-14 2014) (author’s underlining). These definitions are not precise either. As it was discussed earlier, the definition should precisely describe the subject. How much volume change is “appreciable” and how much is not? In addition, cracking condition should be expressed clearly. What kind of cracks is pointed? Do micro cracks considered also? It is evident in the plastic limit determination by thread rolling method that the procedure of crumbling of soil thread starts with appearance of many micro-cracks that will propagate by working on the soil thread until it is crumbled. Although the criterion for detection of soil thread crumbling is vague for different soil types, it is clear that the soil thread has many micro-cracks before reaching to the plastic limit state of soil according to current standard methods. For instance BS EN 1377-2 (1990) states to create a soil ball for plastic limit determination and before beginning to make soil threads, which the soil is still in plastic state and far above plastic limit “Mould the ball of soil between the fingers and roll it between the palms of the hands until the heat of the hands has dried the soil sufficiently for slight cracks to appear on its surface” (author’s underlining). In addition, how is it possible to determine an “indefinite” deformation in a

definite material by keeping a constant stress in a test? BS EN ISO 14688-1:2002+A1 (2013) states a general definition of plasticity that can be expressed for every state of soil: “property of a cohesive soil to change its mechanical behaviour with change of water content”. Hence, it is not a definitive description for soil plasticity. Consequently, regarding the imprecise definitions of soil plasticity, it is not possible to detect accuracy of current standard methods and other alternative methods for determination of limit states of plastic soil.

It is also important to note that the procedure of different methods are not definitions, but are indications for reaching to a particular state of soil (not necessarily the transitional state of soils at their limit state). None of the laboratory tests are perfect and their results should not be considered as a certain benchmark for future investigations. Unfortunately, many researches have been done to correlate their results with the values obtained by current standard methods in condition that we know the results obtained by current standard methods are not reliable essentially.

Heavily destructed soil samples are not mostly appropriate for the determination of engineering properties of soil materials including soil shear strength. Although Atterberg limit tests are among soil index tests, they are directly and indirectly measuring shear strength of specimens made from just a specific range of particle size of in-situ soil. The procedure of soil specimen preparation for Atterberg limit tests is very destructive that hardly can be called as a specimen.

Since interparticle forces are predominant in fine-grained soils, index tests should be designed for determination of these important forces when their water content varies. These forces depend on many soil parameters such as size and shape of soil particles, environmental condition of soil, morphological and mineralogical of soil aggregates, amount of the clay minerals in soil, soil texture and structure, and soil reinforcements such as fibres entanglement in organic soils. The cohesive soil index tests should be designed in a way that these parameters get involved in the test procedure with the lowest possible disturbance. Unfortunately, current standard test methods and alternative proposed tests for determination of soil plasticity do not properly involve these important soil parameters in their test procedures. However, considering the meanings of liquid and plastic limits of cohesive soils in quantifying a qualitative phenomenon, none of the proposed tests are wrong. Yet, it is very important to know the results obtained by each method are reliable and close to the actual liquid and plastic limit states of cohesive soils. Unfortunately, current standard methods and many alternative proposed test have determined a few soil parameters, mostly soil shear strength, as the representative of liquid and plastic limits of soils that have led to results far away from real liquid and plastic limits of soils in some cases. The complex behaviour of different soils with varying water

content cannot be measured by evaluating one or even a few soil parameters in soil tests.

Technically, adsorbed water which has the most influence on cohesion properties of soil cannot be measured by current standard methods. The thickness of adsorbed water is a function of the amount of negative charge on soil particle surface and the positive end of water molecules. The more the negative charge of soil particle surface, the thicker absorbed water layer around the soil particle. It means that the greater activity soil has the more adsorbed water that has not taken into account in water content determination. However, increasing the temperature of the oven will also affect the solid part of soil which will lead to inaccurate water content determination. Although this can be a topic for further investigations, it is not the subject of the current research project.

In addition, chemistry and pH of water used for preparing soil paste in Atterberg limit tests significantly influence the results. Cation exchange capacity of soils is affected by pH of distilled water used in tests might be very different with the pH of water of soil's site. This will certainly affect the test results. However, usage of distilled water might be the most practical way for doing common index tests including Atterberg limit tests.

Although fall cone method has fewer uncertainties in comparison to Casagrande percussion cup method for determination of the liquid limit state of cohesive soils, Casagrande method is the standardised method and very close to what Atterberg introduced for the first time. However, both of these methods assume that a very small and specific undrained shear strength would be the indication of the liquid limit state of soils. It has been investigated by many researchers that all soils do not have a specific shear strength at their limit states and these tests have not used a correct assumption. Casagrande (1932) tried to standardise the Atterberg limit tests for geotechnical engineering utilisation. The problem initiated from the point of quantifying a qualitative phenomenon. The problem became more serious when only one parameter, like soil shear strength, was measured and became an indication for determination of a complex system. Maybe the selection of shear strength as a criterion for determination of Atterberg limits was because of this fact that this parameter is among the most important parameters in geotechnical engineering perspective. However, determination of a single parameter cannot be representative of the whole behaviour of a soil. Hence, although test methods probably had been established better than Atterberg original tests, the approach has been used by geotechnical engineers for determination of Atterberg limits have not been correct. That is why the results obtained by different methods are different for various soil types.

Author's suggestion for solving the problems related to the determination of soil plasticity by conventional methods is utilisation of qualitative research. The reason is that the

behaviour of soil in different states is related to many parameters that form the overall behaviour of soil when its water content varies. It should be noted that many different parameters affect the behaviour of soil and combination of all parameters affects the soil consistency as shown in Figure 2-28. The reason for different values obtained by different methods is that current standard methods involve some of the factors or a portion of some factors in the determination of soil consistency limits. Hence, it is not appropriate to determine one particular property (i.e. soil shear strength) for determination of soil plasticity. Although one particular parameter of soil can vary among different soil types, the overall behaviour of soil should be the same at specific consistency states.

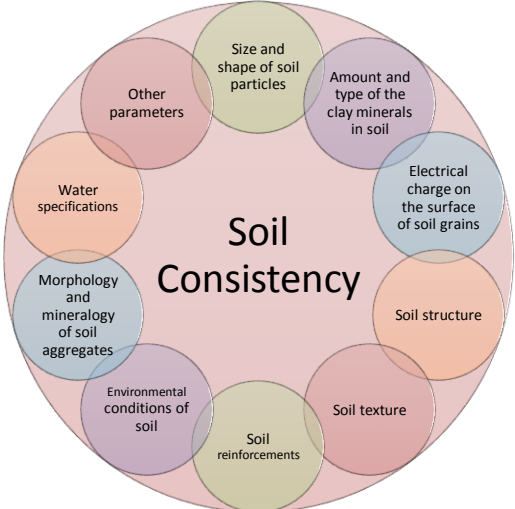


Figure 2-28: Effective parameters on soil consistency states. Note: share of each parameter is variable for each soil

For clarifying the proposed methodology, it is worthwhile to consider an example to show the difference between quantitative research approach and qualitative research approach. Assuming that in a civil engineering department, school director wants to find out whether the output of the department will be good civil engineers or not. He assumes that if the student’s overall average grade at the end of his/her studies be greater than a specific grade, he/she might be a good civil engineer (until now, it might be a good criterion but it is evident that it is not a perfect criterion!). Director’s assistant says the determination of overall grade for each student takes too much time and cost. (Students have to pass 4 years and many classes). He says, based on my experience, students who have a high overall grade, had passed “Static Analysis Course” with high scores. So, let’s just check students’ grades in this course. If they got a score more than a specific value, they will be a good civil engineer probably (This might not be a bad criterion if it worth the time and expenses, but it is a worse criterion than the first one!!). Director’s second assistant says, my colleague might be right, but we still can reduce the time and

expenses. Based on my survey, a student who had answered this specific “Equilibrium Equation Question” in “Static Analysis Course”, he/she has got a good score in this course, hence he/she will be a good civil engineer!!!

The method that these assistants chose for their purpose is **quantitative research approach**. They tried to detect the relationships between variables and generalise the outcomes to the world at large (Van Note Chism et al. 2008). Most of engineers are familiar with this approach. But it is evident that this approach might not be true in many cases especially the method that the second assistant proposed.

The third assistant of the director states the problems of the methods proposed by other assistants. He says that what director looking for is detecting good civil engineers which is a qualitative phenomenon and it is related to many factors. We might not have enough time and budget to consider all the parameters, but our study should cover the main quantitative parameters in addition to the main qualitative parameters and we certainly should not rely on only one parameter. What if the environment of “Static Analysis Course” test was not suitable for test day (temperature, seats, print quality, etc.)? What if the instructor was not good enough for that course? What if the student was sick at that time? What if the student becomes more interested in civil engineering after that course? And many other factors that will certainly effect the goal of our study. He suggests, first of all, we have to make sure that the questions of the “Static Analysis Course” test are appropriately devised then we will index a questionnaire to the test to evaluate other required qualitative parameters. After that, we will combine the results of test and questionnaire and we will analyse them to make sure that a reliable conclusion would be obtained.

The method that the third assistant proposed is a **qualitative research approach**. In this method, he tries to focus on only a few parameters in addition to focusing on the context of the study and finally he recognise himself (researcher) as an instrument (the analyser) of the study (Van Note Chism et al. 2008).

The strong point of qualitative research is its capability to adapt to natural settings which enables exploration of background, reasoning, and other inside parameters that describe the interaction of context and elements in a specific setting (Van Note Chism et al. 2008). It is possible to correlate this example to the subject of this research project. Determination of soil plasticity is the aim of this project which can be correlated to the aim of the director in the example. The method proposed by the second assistant can be correlated to the current standard methods or alternative test methods for determination of liquid limit and plastic limits. The third assistant provides the appropriate solution for the problem which is the utilisation of qualitative research method. The final solution for determination of soil plasticity is also utilising qualitative research based approach. There

are different theoretical perspectives for application of qualitative research in projects namely: positivist, post-positivist, interpretivist, critical, post-modern, and post-structural (Van Note Chism et al. 2008). Important features of the common epistemological perspectives are summarised in Table 2-8.

Table 2-8: Comparison of theoretical perspectives (Koro-Ljungberg and Douglas 2008)

Theoretical perspective	Post-positivist	Interpretivist	Critical/ emancipatory	Postmodern/ poststructural
View on reality	Single falsifiable reality	Multiple subjective realities	Multiple subjective and political realities	Multiple fragmented Realities
Purpose	To find relationships among variables, to define cause and effect	To describe a situation, experience, or phenomenon	To produce a socio-political critique	To deconstruct existing 'grand narratives'
Methods	Methods and variables defined in advance, hypothesis-driven	Methods and approaches emerge and are to be adjusted during study	Methods and approaches designed to capture inequities	Methods and approaches generated during the study
The role of researcher	Researcher is detached	Researcher and participants are partners	Researcher and participants are activists	Researchers and participants have various changing roles
Outcome or research product	Context-free generalizations	Situated descriptions	Critical essays, policy changes	Reconceptualized descriptions of the phenomenon

In this research, it is tried to utilize post-positivist epistemological perspective which assumes that there is an absolute truth (soil plasticity) and data obtained from test (that should have minimum requirements) is consistent with an assumed truth and there is always a possibility of facing a counter-example (Van Note Chism et al. 2008). The focus of this research study is to devise a new apparatus that covers the most uncertainties and deficiencies of conventional methods. In future investigations, it will be tried to solve the main problems of soil consistency determinations including determination of soil water content and effect of the full range particle size distribution of soil on its consistency states.

2.8 Conclusion

Atterberg limit tests are among soil index tests that provide key information about engineering properties of soil for geotechnical engineers. Since the results of index tests are a base for having a vision of overall behaviour of soils in different situations, in addition to correlating the index tests results to engineering parameters of soils, it is very important that the geotechnical engineer can trust the results of Atterberg limit tests. Atterberg limit test methods should be devised in a way that the tests can determine the behaviour of soil by changing only water content of the soil. The test methods should

provide a system that soil gets deformed in the system and the deformation of soil or behaviour of soil can be determined. It is expected that cohesive soils should have the same behaviour at specific limit states according consistency definition. The new determination methods which are going to be proposed, in addition that should not have the past deficiencies, should be reliable, easy, quick and inexpensive at the same time. Unless, they will not be replaced by current standard methods.

It is of great concern to consider the most important factors of soils affecting the behaviour of soils (i.e. size and shape of soil particles, electrical charge on the surface of soil grains, environmental condition of soil, morphological and mineralogical of soil aggregates, amount and type of the clay minerals in soil, soil texture and structure, soil reinforcements, and etc.) in determination of Atterberg limits of cohesive soils. Determination of just one engineering parameter (i.e. shear strength of soil) might cover some of the factors and provide good results for some soils, but it will certainly provide misleading results for other complex soils such as organic soils. Covering all or even some parts of problems stated related to Atterberg limit tests might significantly affect the test results which can lead to significant changes in many empirical correlations established based on current Atterberg limit tests. There is a very meaningful proverb in Persian that states: “خشت اول گز نه‌د معمار کج، تا ثریا می‌رود دیوار کج” which means if an architect puts the first brick of a wall inclined, the wall will be built inclined to the sky. Unfortunately, many researches are based on the result of current standard methods and incorrect assumptions over the last century. In addition, all the presented empirical correlations are based on the results of conventional methods which are affected by the issues discussed in this chapter. Covering the problems of conventional methods may lead to different values for limit states of soils. Although the concepts will not be altered, the equations and results will be different. This will indirectly affect the empirical correlation and even soil classification systems. However, this is the path of evolution in science. It should be noted that although current standard methods have several problems, they are working widespread with a degree of reliability. But it should be considered that this degree of reliability is not enough for all geotechnical applications, especially for soil classification.

A schematic figure of current situation, proposed study, and future researches of soil consistency determination is shown in Figure 2-29. This figure briefly demonstrates contents covered in this Chapter and proposed researches for staged solutions regarding soil plasticity determination. Inappropriate conditions of current methods (such as sample preparation, intrinsic apparatus deficiencies/limitations, operator's performance, water content determination, and etc.) have explained in this chapter. It had also tried to

clarify inappropriate approach (quantitative approach) for determination of a qualitative phenomenon. Combination of these two main items has led to inconsistent results obtained by different methods. In this research, it is tried to improve some aspects of test conditions for determination of liquid and plastic limits of cohesive soils. It is tried to define a mechanical soil deformation system and establishing an easy, inexpensive, and repeatable test method following qualitative approach for determination of plastic state ('very stiff' consistency limit) and liquid state ('soft' consistency limit). The proposed apparatus and test method will be described in next chapter. Current research will provide a proper research base for later comprehensive study covering the deficiencies pertinent to the determination of soil consistency limits.

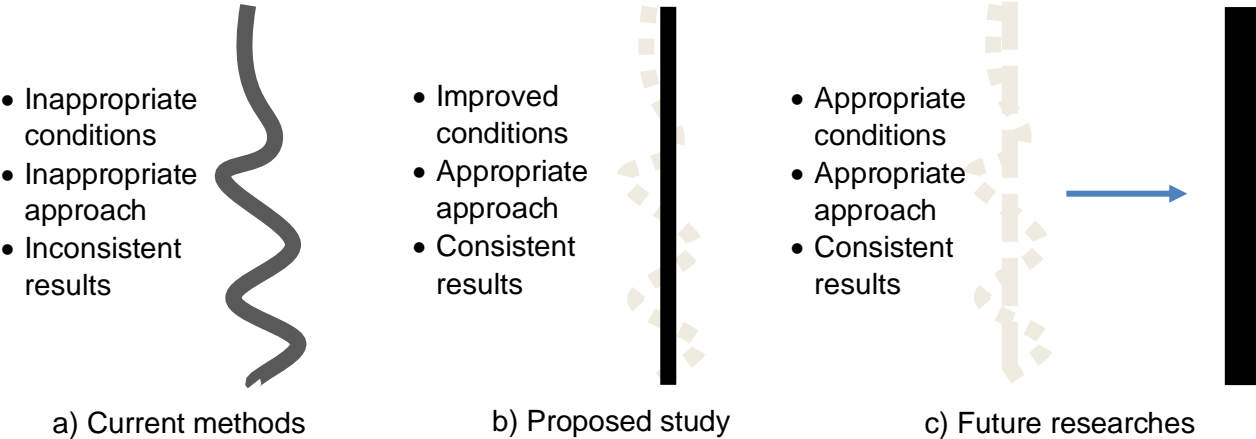


Figure 2-29: Schematic figure of current methods, proposed study, and future researches of soil consistency determination

3 Manafi's Apparatus and Test Method

As it was discussed in chapter 2, Atterberg (1911) proposed several simple and inexpensive tests for determination of soils consistency limits. Later his methods were utilised in geotechnical engineering by Karl Terzaghi and Arthur Casagrande. Their investigations had led to standard procedures for determination of Atterberg limits. The standard methods have several problems that many researchers proposed alternative test methods for determination Atterberg limits. It was described that in the standardisation of Atterberg's original tests for geotechnical engineering applications, quantitative approach has been used for the determination of consistency limits which is different with the original qualitative approach of Atterberg. Researchers with quantitative approach have tried to detect the relationship between a specific parameter of soil and its consistency limits. However, soil consistency is a qualitative phenomenon and it is required to observe the behaviour of soil when its moisture content varies.

The new proposed apparatus and test method for determination of soil consistency are devised based on the nature of soil deformation. It provides a system that soil will get deformed and it is possible to determine the rate of deformation in the system. In other words, a defined system of soil deformation will be correlated to specific soil consistencies (liquid and plastic limits). Hence, the soil deformation process affected by the most important parameters in soil consistency can be quantified by a simple measurement of deformation time. In the proposed method, considering the consistency definition discussed in chapter 2, it is assumed that although different soil types might have different values for various parameters (i.e. various shear strength at consistency limits), they should have the same overall behaviour at various consistency limit states including liquid and plastic limit states. It should be noted that specimen preparation for Atterberg limit tests has several issues that have been discussed in chapter 2, yet this approach is able to cover those deficiencies too. Therefore, utilising this approach, the soil plasticity can be calculated more reliably in comparison to the current standard methods.

The key question is that: what system is capable of deforming soil in a way that the whole procedure be reliable, easy, quick and inexpensive at the same time? If the new proposed method does not have these criteria, it will not be an ideal test method to be replaced by current test methods. Several methods had been considered and finally an extrusion method was selected due to its good compatibility to the requirements of proposed approach. The evolution of apparatus design is described in "3.6 Manafi's apparatus design evolution" section.

Although extrusion method has not been used in geotechnical engineering extensively, it has been utilized for more than 100 years in other industries including metal industries, food industries, polymer industries, pharmaceutical industries, package industries, and etc. While the concept of extrusion is the same, each industry has its own requirements and various techniques are used to satisfy their own needs. Mechanical and theoretical analysis of extrusion method have been well established in different industries (i.e Lee et al. 1977; Dieter and Bacon 1988; Lee 2000; Saha 2000; Guy 2001; Michaeli 2003; Giles et al. 2004; Bauser et al. 2006; Hosford and Caddell 2011; Moscicki 2011). Although objectives of the current research are different with industrial products utilising extrusion method, concepts and fundamentals of extrusion technique are the same and will be discussed in “3.2 Extrusion method” section.

As it was discussed in chapter 2, extrusion method had been tried to determine soil consistency limits. However, the approaches of using this method for determination of soil consistency limits had not been right. It has been tried to correlate a specific extrusion pressure to a particular undrained shear strength or to the results obtained by Atterberg limits conventional methods. In contrast, the approach of the current study is the determination of overall behaviour of soil utilising extrusion technique.

3.1 Apparatus design concepts

One of the main questions that should be considered for the design of an apparatus based on proposed approach is: how is it possible to measure the behaviour of soil when its moisture content varies? For this purpose, it would be appropriate to have a look at the meaning of “behaviour” and look for a proper scientific way for its measurement. Oxford Dictionary of English defines the word “behaviour” as: “the way in which one acts or conducts oneself, especially towards others” and provides more information with “the way in which an animal or person behaves in response to a particular situation or stimulus” and “the way in which a machine or natural phenomenon works or functions” (Stevenson 2010). These explanations show that the reaction of soil to a specific condition exerted on the soil is the behaviour of soil to that condition. All soils will react to the condition defined by the test method, but that test might not be devised to measure the behaviour of soil. For instance, determination of only one or several parameters of soil (i.e. soil shear strength in fall cone test) is not the determination of overall soil behaviour during the test. Moreover, that test might not have been defined well to measure the behaviour of soil properly. For instance, variable test conditions provided by different operators in Casagrande persuasion cup and bead rolling tests. Hence, the key points in the measurement of the behaviour of soil are:

1. Providing a well-defined situation that soil can be deformed or react to that situation while the situation is repeatable for many times with the least possible difference.
2. Overall soil behaviour affected by all or the most important factors could be measured during the test.

3.1.1 Workability

One of the ways for determination of soil behaviour is determination of soil workability. Workability has different meanings in different areas and it is necessary to look at it in the context. In general, a workable system is practical and effective; and a workable substance can be shaped by hand (Longman Group 2011). From this definition, it can be understood that the workability of a system can be scaled by measuring the efficiency of the system for reaching to the purpose of a particular job from different aspects such as practicality of the project, quality and financial aspects of the final product, and etc.

Workability for building materials in civil engineering projects is: “the ability of a mortar or a fresh concrete to fill correctly a mold or a formwork thanks to a well-studied batching of its constituents that give him a sufficient fluidity without harming its strength and its homogeneity” (Kurtz 2004). Building materials such as concrete are usually required to fill a mould with specific geometry to shape a structural element. Hence, it is required to have a mortar with proper consistency to fill the mould considering the shape and reinforcements of the element having a specific strength. This process is not usually considered in the case of soil material. The consistency of unhardened concrete can be monitored by slump cone test as described in ASTM C143 / C143M-15a (2015).

Workability in metalworking process has a different purpose which is not expected in geotechnical engineering projects. It is of importance that the metal gets deformed without formation of local necking (plastic instability) or cracks (Dieter and Bacon 1988). Workability in steel industry is a complex technological concept due to different requirements of steel elements in various applications. Particular details of the deformation process are of great concerns as well as ductility of steel elements (Dieter and Bacon 1988).

Workable range of water content in ceramic industry and white-ware production is usually used for a clay with a moisture content below the sticky limit and above plastic limit (Wesley 2014). Workability range of water content in agricultural industry will be defined based on the efficiency of agricultural machinery in ploughing and tilling a clay soil, and soils with moisture content less than plastic limit are usually deemed for workable soils (Barnes 2013).

UFC 3-220-04FA (2004) considers workability as an important factor for choosing backfill materials in geotechnical projects that is explained as the ease with which the soil can

be placed and compacted. Workability of soil materials used in backfill construction is affected by characteristics such as (UFC 3-220-04FA 2004):

- ease of modifying water contents in the field by increasing water content with wetting or decreasing water content with aeration
- sensitivity to the compaction considering optimum water content
- amount of compaction effort for reaching to predefined densities

Since backfill materials are usually coarse grain soils and compaction of soils is usually considered for soils in unsaturated states, it shows that workability concept is different in various situations for different soil types even in geotechnical projects.

Considering the cases expressed in this section, the concept of workability should be defined based on the purpose of the project and its requirements. Since Atterberg limit tests are among soil index tests that usually characterise the soil material at site, the new proposed method for determination of workability of soil should concentrate on the behaviour of soil when a specific work is done on the soil specimen. In this regard, it is required to have a look at the meaning of ‘work’ in physics science.

3.1.2 Physical concepts

The word “work” has several meanings in different contexts just like the word “workability” that had discussed. Since it is decided to do a physical action on soil and measuring the soil behaviour to that specific physical action, it is required to know the meaning of work and the way of its measurement in physics science. “In physics, work is done only if an object is moved through some displacement while a force is applied to it” (Serway et al. 2006). Formula for calculation of work done on an object by a constant force as shown in Figure 3-1 is (Serway et al. 2006):

$$W = (F \cos \theta)\Delta x \tag{3-1}$$

where F is the magnitude of the force, θ is the angle between the directions of \vec{F} and $\Delta\vec{x}$, and Δx is the magnitude of the displacement.

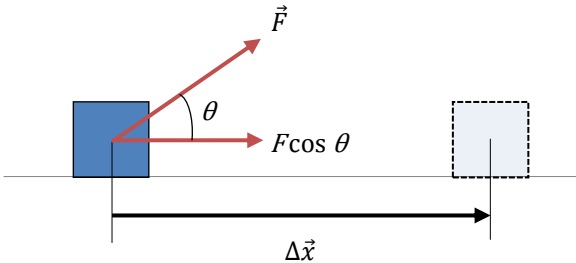


Figure 3-1: Illustration for work done on an object by a constant force

Considering the purpose of this study regarding the determination of soil consistency and having a look at the meaning of consistency discussed in chapter 2, it is required to

observe the behaviour soil over a period of time. As it can be seen, there is no time parameter in the formula of work. Work does not depend on time and therefore does not reflect the amount of time that the force acts on the object causing the displacement. In addition, there is no direct relation to different material of the object in the formula of work. However, it is evident that different objects might need a different amount of force to overcome the resistance against movement of the objects as it is demonstrated in Figure 3-2. In this figure, the work done by F_1 is constant for both of objects in Figure 3-2 (a) and (b), but the work had done by resistant force in Figure 3-2 (a) is bigger than Figure 3-2 (b). the difference between the net work done on objects in Figure 3-2 (a) and (b) causes faster movement of object in case (b) than the object in case (a) in this figure. Actually, the net force in case (b) of Figure 3-2 ($F_1 - F_{r2}$) is bigger than case (a) ($F_1 - F_{r1}$) and this will lead to more acceleration of object in case (b) by assuming an identical mass in both cases according to Newton's second law of motion ($F = m \cdot a$; F is the force, m is the mass of object, and a is the acceleration of object). Hence it is possible to measure the effect of resistant force by measuring the rate of work done on the object. Therefore, it is appropriate to introduce another physical concept with title of “power” that quantifies the rate of work.

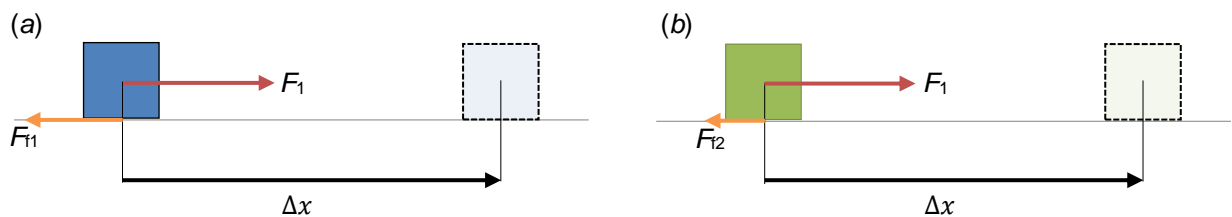


Figure 3-2: Different resistant force for various object materials. (a) An object with high resistant force against movement. (b) An object with low resistant force against movement

The work done by force F_1 on both of objects in Figure 3-2 (a) and (b) is equal, but the work in case (b) has done quicker due to less resistant force as explained. The rate at which work is done is called **power**. The average power can be calculated by the following formula (Serway et al. 2006):

$$\bar{P} = \frac{W}{\Delta t} = \frac{F\Delta x}{\Delta t} = F\bar{v} \quad (3-2)$$

where W is the work, Δt is the time interval, F is the magnitude of the force in direction of displacement, Δx is the magnitude of the displacement, and \bar{v} is the average speed of the object.

Considering the physical concepts explained in this section, if there is a specific driving force and a variable resistant force, it is possible to calculate the work of resistant force by measuring the power of that specific force. The resisting force in this research is the

force that does work to deform the soil specimen and will be discussed later in details based on the design of Manafi's apparatus.

3.2 Extrusion method

Since extrusion method is chosen for deformation of soil paste in this research, it is appropriate to consider the mechanisms and theoretical analyses of the extrusion process. Extrusion method is a well-established method in many industrial products. However, each product has its own requirements and various techniques are used to satisfy the needs by utilising special technology to work with various materials which require special design and construction of extrusion presses and extrusion tooling. However, in current study only measurement of deformation rate of soil paste in extrusion process is important that might not be of great concern in other industries. In this research, it is tried to detect some specific states of cohesive soils by providing a particular condition that cohesive soils should behave similarly in that situation. Utilisation of extrusion method is only for providing that particular situation which enables the researcher to observe and measure the deformation of soil in a controlled system. The purpose of this research is to find a property of soil (soil consistency) utilising extrusion method, but the purpose of usage of this method in other industries is to produce appropriate productions efficiently. Hence, although the principles of the method are the same, the purpose and design plans are different.

3.2.1 Introduction to extrusion method

The term of "extrusion" is defined in every industry based on the requirements of process or products of that industry. For instance, in the food industry, "extrusion cooking" is defined as: "a process of mechanical and thermal transformation" (Guy 2001). A close meaning of extrusion for geotechnical applications and especially for this research project can be found in metallurgy science. Extrusion has a long history in metal forming and is defined as: "a plastic deformation process in which a block of metal (billet) is forced to flow by compression through the die opening of a smaller cross-sectional area than that of the original billet" (Saha 2000). A schematic figure of extrusion process is shown in Figure 3-3.

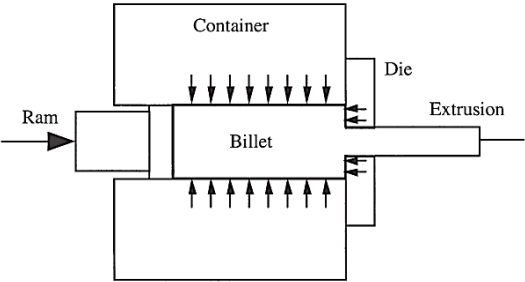


Figure 3-3: Extrusion process (Saha 2000)

During the extrusion process, the billet is under direct and indirect compression forces which deforms and passes through the die orifice. Extrusion in the metal industry can be done by hot or cold methods based on the alloy and requirements of the product. However, hot extrusion method facilitates plastic deformation due to lower deformation resistance of the metal in higher temperatures (Dieter and Bacon 1988). Hence, the force required for the deformation process in hot extrusion method is less than cold extrusion method. Although billet temperature has a great influence on the design of extrusion equipment in the metal industry especially in the design of die and container, it is not effective in apparatus design for determination of soil consistency in room temperature. However, the temperature has effect on water viscosity that might affect the soil consistency, but it has not a great influence for considering a special equipment design for the project purpose. In addition, Casagrande (1932) had done a few careful Atterberg limit tests and observed no difference between the values of tests at 20 °C and 35-40°C. However, the effect of temperature can also be studied in the next stage in the comprehensive study for determination of soil consistency which is not among the objectives of the current research.

General usage of extrusion method in metal industry is for production of cylindrical bars or hollow tubes. Irregular cross sections can also be formed by extrusion method (Dieter and Bacon 1988). However, shape of cross section of extruded soil paste is not of importance in this research project.

3.2.2 Classification of extrusion processes

In general, two basic types of extrusion is used in metal industry: direct, and indirect (reverse) extrusion methods. Selection of these two methods in metal industry is based on the requirements of the final product and technological equipment.

3.2.2.1 Direct extrusion method

One of the most common methods in the extrusion of materials is direct extrusion method. Schematic figure of direct extrusion method is shown in Figure 3-3 which the material is placed in the container and is pushed by a rammer to get deformed and pass through the die orifice. The reason for naming this method with the *direct method* is that the direction of material extrusion is the same as the direction of the rammer when the material extrudes. Since the billet has a relative motion to the wall of the container, there will be a frictional force resisting the relative movement. This frictional movement depends on the materials of billet and container wall which may have a high value if the length of the billet is long or the frictional coefficient between billet and container wall is

high that might be due to great extrusion pressures. These situations might happen in industrial practice such as metal extrusion.

3.2.2.2 Indirect extrusion method

Another method for extrusion materials is the indirect method (also inverted, or back extrusion method). In this method, the material is placed in the container and the die, which is connected to a hollow stem, moves relative to the container. In this method the direction of material extrusion opposite of the direction of the rammer when the material extrudes. Schematic figure of indirect extrusion method is shown in Figure 3-4.

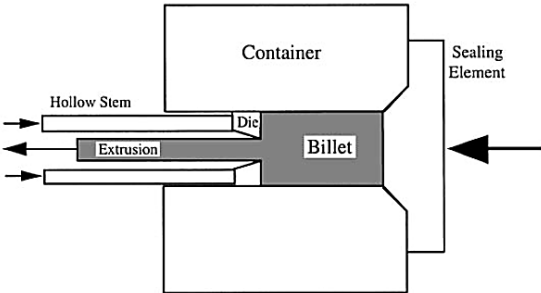


Figure 3-4: Extract from Saha (2000). Indirect extrusion method

Since there is very little movement between walls of container and billet, negligible frictional resistance is produced. This is an advantage of indirect extrusion method in comparison to direct extrusion method in case of existence of high frictional force. However, direct extrusion method is simpler and there is no need to have a hollow stem that the extruded material has to pass through it. In addition, the friction force is directly related to the amount of contact surface between billet and container which in the case of small billets is very low. A typical curve demonstrating the relation between ram travel and extrusion pressure for direct and indirect metal extrusion is shown in Figure 3-5.

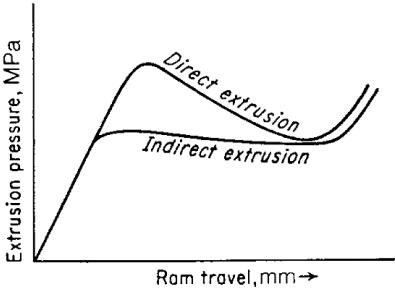


Figure 3-5: Extract from Dieter and Bacon (1988). Typical curves of extrusion pressure against ram travel for direct and indirect metal extrusion

It should be noted that the typical graph shown in Figure 3-5 depends on many factors explained in “3.2.3 Mechanics of extrusion”. The general curve might be very different in the extrusion of different materials in different design conditions. For instance, long billets

in extremely high pressures lead to high frictions between billet and container wall that reduces gradually by the extrusion process as the length of billet decreases. That is why a peak point and reduction in extrusion pressure is observable in Figure 3-5 for direct metal extrusion. Hence, the peak point is not observable for extrusion of plastic materials with low frictions due to low extrusion pressure and short billets. The technical points are discussed in details in next sections.

3.2.3 Mechanics of extrusion

Extrusion method is a well-established method in metallurgy science and metal forming industry. This method has been used for more than 100 years and mechanics of this method has been investigated comprehensively. However, metal forming has its own circumstances that affect the mechanics of extrusion during the extrusion process. For instance, thermodynamics and heat transfer to extrusion material is of great concern in metal forming. In addition, investigation of plastic deformation in industrial material forming is necessary to achieve the desired geometry with specific mechanical properties. However, in this investigation, it is only intended to observe the behaviour of cohesive soil in a controlled system provided by the extrusion process.

Mechanics of extrusion is explained in metallurgy textbooks in details (i.e. Dieter and Bacon 1988; Lee 2000; Saha 2000; Giles et al. 2004; Bauser et al. 2006; Hosford and Caddell 2011). Related topics will be discussed in this chapter to understand the process of soil deformation inside proposed apparatus. The textbook “Aluminum Extrusion Technology” written by Saha (2000) is a very good reference for the theoretical explanation of extrusion process that is used in this section for explaining the mechanics of extrusion and reader can refer to it for further information.

3.2.3.1 Plastic deformation and material flow

Application of plastic theory helps analysis and prediction of plastic deformation of materials in the extrusion method. Depending on the various approaches of forming operations, different mathematical equations can be written (Saha 2000).

Considering a simple homogeneous (uniaxial) compression, if the stress (σ) reaches to flow stress ($\bar{\sigma}$), the material will flow plastically. Plastic flow of materials in extrusion method depends on many factors (Saha 2000):

- Material properties
- Interface friction between material-container and material-die
- Extrusion ratio

Different flow patterns can be observed during extrusion process depending on various flow characteristics of materials. Schematic flow patterns for typical extrusion methods is shown in Figure 3-6.

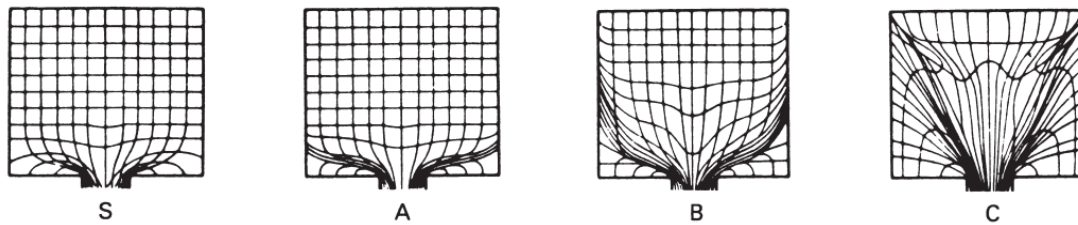


Figure 3-6: Extract from Saha (2000). Schematic patterns of four different flow types in extrusion method (Laue and Stenger 1981)

If the material is homogeneous (uniform properties in both longitudinal and transverse directions) and interface frictions be almost zero (considering fully lubricated conditions), pattern S can be expected for material extrusion (Saha 2000).

If the material is homogeneous and die interface friction is negligible but there is container-billet interface friction, pattern A can be expected for material extrusion. In this pattern, the material at the centre of the billet extrudes faster than the peripheral material. There is also a dead-material zone in the corner of the leading end of the billet between the container wall and the die face (Saha 2000).

If the material is homogeneous and die interface friction and container-billet interface friction are not negligible, pattern B can be expected for material extrusion. In this pattern, the dead-material zone is more extended and more shear deformation will be occurred in comparison to flow pattern A (Saha 2000).

If the material is not homogeneous or temperature distribution in the billet is not uniform, pattern C can be expected for material extrusion. In this pattern, the dead-material zone is more extended and more shear deformation will occur in comparison to previous flow patterns (Saha 2000).

Many factors affect material flow and properties of extruded material. Some of the most influencing factors are (Saha 2000):

- Type of extrusion (i.e. direct or indirect extrusion method)
- Pressure load
- Size and shape of container
- Size and layout of die
- Surface frictional effects
- Length of billet and type of material
- Billet and container temperature
- Die and tooling temperature
- Extrusion ratio

- Extrusion speed

3.2.3.2 Dead-material zone

Generally, material shears along the face of the dead-material zone in the corner of the leading end of the billet between the container wall and the die face (Saha 2000). This zone may be considered as a conical die surface that material extrudes over it. A cross section of an aluminium alloy butt remaining after extrusion is demonstrated in Figure 3-7(a). Schematic cross section diagram of the dead zone is also demonstrated in Figure 3-7(b).

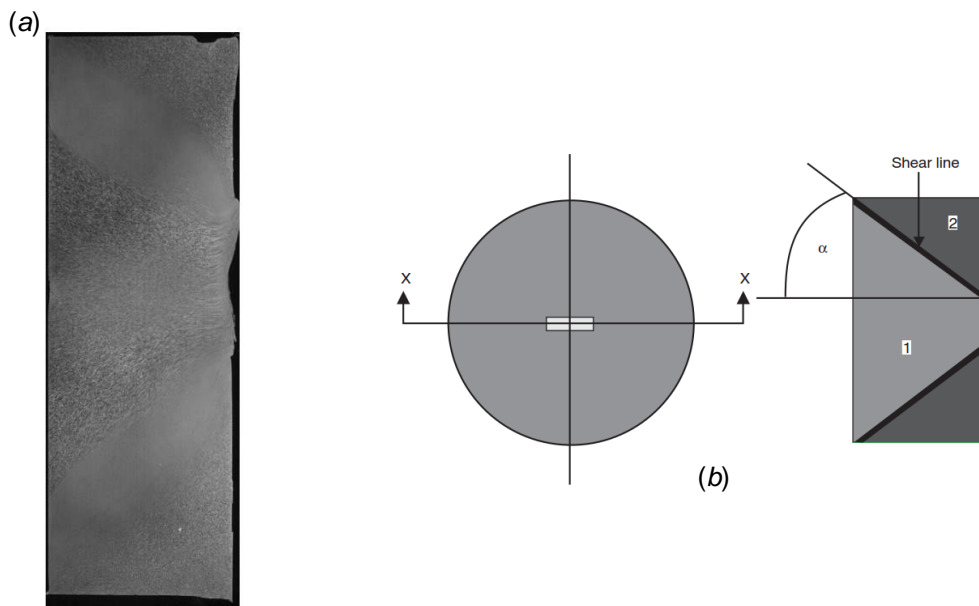


Figure 3-7: Cross section of butt after extrusion. (a) Typical etched cross section of an alloy butt. (b) Schematic cross section diagram of dead zone (Saha 2000)

Zone 2 in Figure 3-7(b) is the dead-material zone that billet gets deformed and extrudes over it. Angle of the dead material zone (α) is a function of several parameters (Saha 2000):

$$\alpha = f(ER, \bar{\sigma}, m, m') \quad (3-3)$$

where ER is the extrusion ratio (cross-sectional area of the container (A_C) / cross-sectional area of extruded material (A_E)), $\bar{\sigma}$ is the flow stress, m is the friction factor between billet and container interface, and m' is the friction factor between flowing material and die-bearing interface.

The dead zone is important in industrial products because this zone is fairly static during the extrusion process and is prone to oxidation and other defects that if enters to the extruded products might have a harmful effect. Hence, it is tried to stop extrusion at a safe margin zone and keep an adequate amount of butt to avoid having an oxidation or other inclusions in the final product (Saha 2000). The relationship between dead zone

and butt thickness is shown in Figure 2-10. Based on industrial practice, 10 to 15% of the billet length is considered as butt thickness in metal extrusion by direct extrusion method (Saha 2000). However, quality of material in the dead zone is not an important case in this research because it is not expected that the properties of soil paste in the dead zone are different from properties of the billet in a short period of time of test and there is no final commercial product.

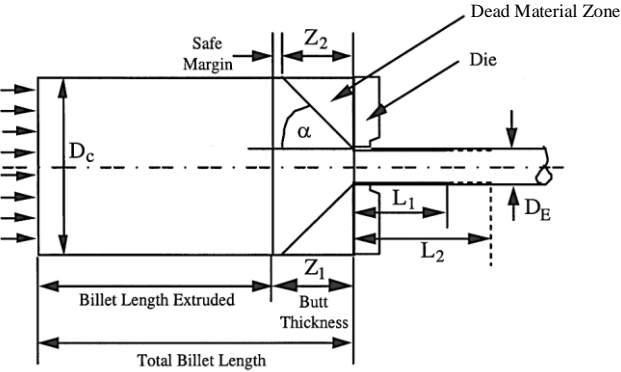


Figure 3-8: Relationship between dead zone and butt thickness (Saha 2000)

In addition, extrusion of material in the dead zone might require more extrusion force and even some material might remain in the container. It is possible to increase the number of dies and distribute it properly to reduce the volume of dead zone material as it is shown in Figure 3-9.

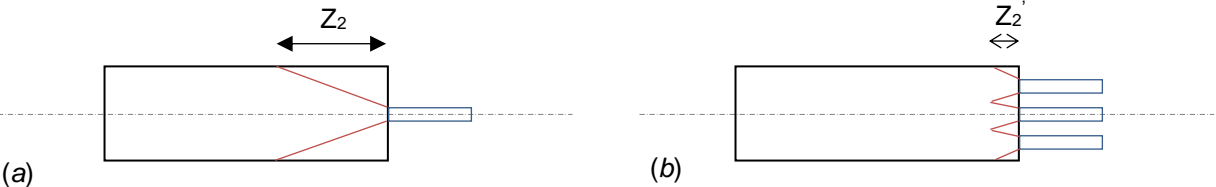


Figure 3-9: Change in formation of dead zone material by increasing number of dies. (a) Extrusion with one die. (b) Extrusion with several dies.

3.2.3.3 Plastic strain and strain rate

It is required to define strain or deformation and strain rate or deformation rate to study material plastic flow. In this case, the initial condition cannot be considered and instantaneous length change should be considered for determination of length change. Hence, natural (effective) strain is defined by:

$$d\bar{\epsilon} = \frac{dl}{l} ; \quad \bar{\epsilon} = \int_{l_0}^l \frac{dl}{l} = \ln \frac{l}{l_0} \tag{3-4}$$

where, l_0 is the initial length, and l is the final length of deformed material. It is possible to consider volume constancy relation in extrusion process:

$$Al = A_0l_0 \quad (3-5)$$

where A is the final area of deformed material, l is the final length of deformed material, A_0 is the initial area, and l_0 is the initial length. Hence natural strain equation can also be written based on volume constancy relation:

$$\bar{\epsilon} = \ln \frac{l}{l_0} = \ln \frac{A_0}{A} \quad (3-6)$$

where l is the final length of deformed material, l_0 is the initial length, A_0 is the initial area, and A is the final area.

Considering the equation of natural strain in the case of extrusion method with circular cross-sections, the natural strain equation can be rewritten as follows:

$$\bar{\epsilon} = 2 \ln \frac{D_C}{D_E} = 2 \ln \sqrt{ER} \quad (3-7)$$

where D_C is the inside diameter of the container and D_E is the equivalent diameter of the extruded material, and ER is the extrusion ratio.

In the extrusion process, the material passes through deformation zone that undergoes a rapid acceleration and a complex flow pattern. Usage of mean strain rate can help for determination of flow stress in the extrusion process. Figure 3-10 shows a simplified conical deformation zone which equation for the length of the deformation zone can be expressed as (Saha 2000):

$$L = \frac{D_C - D_E}{2 \tan \alpha} \quad (3-8)$$

where D_C is the inside diameter of the container, D_E is the equivalent diameter of the extruded material, and α is the angle of the dead material zone.

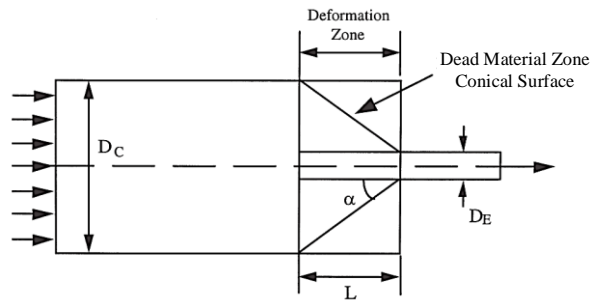


Figure 3-10: Material geometry in extrusion method (Saha 2000)

The equivalent diameter of the extruded material can also be written as (Saha 2000):

$$D_E = \frac{D_C}{\sqrt{ER}} \quad (3-9)$$

where D_C is the inside diameter of the container, and ER is the extrusion ratio.

Hence, the mean effective strain rate can be written as (Castle and Sheppard 1976; Castle 1993):

$$\dot{\epsilon} = \frac{6VD_C^2 \tan \alpha}{D_C^3 - D_E^3} 2 \ln \frac{D_C}{D_E} \tag{3-10}$$

where V is the average ram speed, D_C is the inside diameter of the container, α is the angle of the dead material zone, and D_E is the equivalent diameter of the extruded material.

3.2.3.4 Friction models

Tribology (friction, lubrication, and wear) of die/material interface is of great importance in the extrusion process for having a product with high-quality surface and shape conditions. All components that affect the friction between material and extrusion device are of great concern when the billet length is long and quality of the product is important. These components also depend on the type of extrusion method. Schematic friction force components in direct and indirect extrusion method with flat-face dies are shown in Figure 3-11 and Figure 3-12 respectively. Material flow in a lubricated direct method with flat-face die might be very similar to indirect process (Saha 2000). Hence, the typical curve for direct extrusion method in Figure 3-5 would be similar to indirect extrusion method.

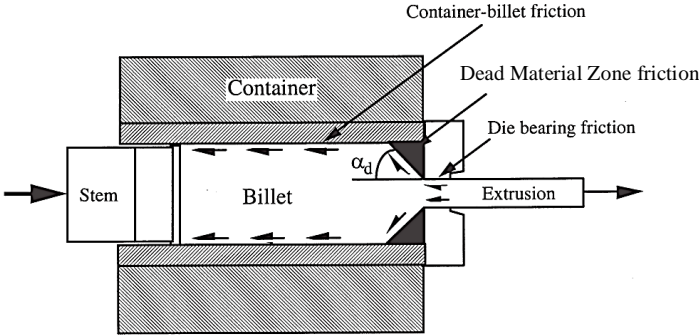


Figure 3-11: Friction components in direct extrusion method (Saha 2000)

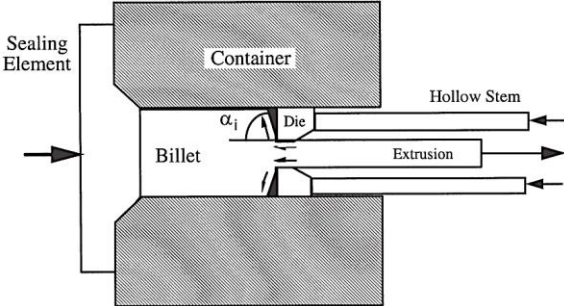


Figure 3-12: Friction components in indirect extrusion method (Saha 2000)

The friction force is a resistance force against relative movement of two materials, which are in contact with each other. Friction force can be expressed as (Amontons 1699):

$$F_F = \mu N \quad (3-11)$$

where μ is the friction coefficient, N is the normal force. This simple model provides well results in condition that relatively light interface loads are involved and surfaces contact only at occasional asperity peaks. For bulk deformations that are the result of intimate contact with very high interface loads, questionable results may be obtained by this model (Saha 2000). This condition can be expected in extrusion process in the metal industry. However, in the current study, there is neither long billet of soil paste nor high interface loads as they are in metal deformation by the extrusion method. For low interface loads, it can be simplified as the force required for pushing a soil specimen through a hollow tube. Although this force varies for different soil materials, the amount of interface friction for soils would be within a short range for low interface areas. Hence, it can be assumed that soils have a small range of friction between soil and container that can be ignored in comparison to the large amount of force required for the overall deformation of soil in the proposed apparatus. In addition, Kayabali et al. (2016) has provided a general graph for extrusion pressure at failure for soils with various water contents (Figure 3-13). This figure shows a fairly constant extrusion pressure at failure in a direct extrusion device which represents a very small interface friction, because if it was not, a peak point and a decreasing extrusion force would be observed as it might happen in very high extrusion pressures or non-homogenous materials and long billets like what is demonstrated in Figure 3-5 for metal deformation.

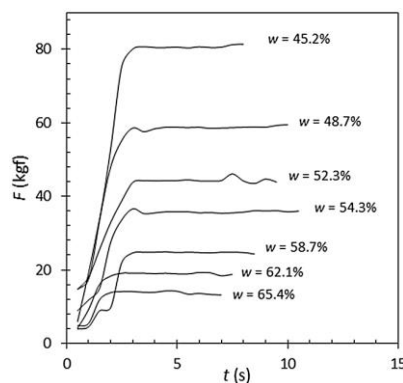


Figure 3-13: Extract from (Kayabali et al. 2016). Extrusion force diagram for mud press machine

If the contact pressure between container and billet increases, the real contact surface will increase too. This can be seen in Figure 3-14. Bowden and Tabor (1964) state that by using adhesion theory, the friction force is directly proportional to the real area of

contact. In condition of very high pressures, the real area of contact (A_R) becomes very close to the apparent area of contact (A_A).

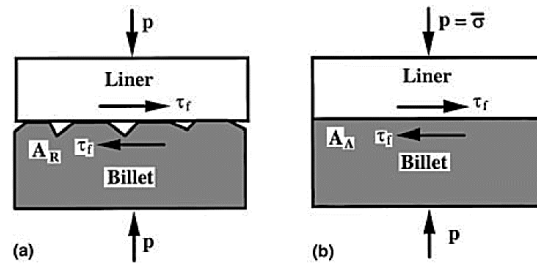


Figure 3-14: Friction model in direct extrusion process. (a) $A_R < A_A$. (b) $A_R = A_A$, $p = \bar{\sigma}$ (Saha 2000)

There are two main friction resistance in direct extrusion method with very high extrusion pressures: billet-container interface friction, and friction between flowing material and the dead-material zone conical interface. Considering a practical point of view, two types of billet-container interface friction can be modelled (Saha 2000):

- Sticking friction (when very high friction is involved)
- Sliding friction (when very low friction is involved)

Since very high pressures are used in metal formation industries while long billets are deformed that results in large interface areas, sticking friction model is considered. In this condition, a friction factor model (stiction model) is proposed for determination of friction force (Shey 1983):

$$F_F = mkA_R \quad (3-12)$$

where m is the friction factor, k is the material shear strength, A_R is the real contact area. If a proper layer of lubrication would be considered, m reaches to about zero.

Friction between flowing material and conical die-like channel formed by the dead-material zone shown in Figure 3-11 is not more than the shear stress of the material. The frictional stress considering a friction factor equal to one can be expressed based on Von Mises yield criteria as:

$$\tau_f = k = \frac{\bar{\sigma}}{\sqrt{3}} \quad (3-13)$$

where k is the material shear strength, and $\bar{\sigma}$ is the flow stress of the material.

Study on die surface after several cycles of extrusion in metal extrusion industry has shown that friction of flowing metal passing through the die is very complex. As a result of strong adhesion between flowing metal and die surface, an adhesive layer of metal will be developed on the surface of die and the die will be worn (Saha 2000). Die worn is due to high pressure, high temperature, and very strong adhesion between die and

metal. These conditions are not expected in soil paste extrusion in the current study. Hence, the sliding friction model is expected for current research due to the low contact surface and low extrusion pressures. However, even in the case of high interface friction, this force will affect the overall behaviour of soil during soil deformation that is going to be determined in this research.

3.2.3.5 Extrusion pressure

Considering the typical pressure curve for direct extrusion method shown in Figure 3-5, the force for overcoming the frictional forces in extrusion process can be attributed to the difference between the maximum and minimum pressures in the graph. However, the total extrusion pressure (P_T) is equal to the sum of pressure required for the plastic deformation of the material (P_D), the pressure required to overcome frictional resistances (P_F), and pressure to overcome redundant or internal deformation work (P_R). The total extrusion pressure can be expressed as (Saha 2000):

$$P_T = P_D + P_F + P_R \quad (3-14)$$

Each term of this equation is a function of several parameters. Deformation pressure is a function of flow stress ($\bar{\sigma}$) and natural strain ($\bar{\epsilon}$). Flow stress can be defined as the following functional form:

$$\bar{\sigma} = f(\bar{\epsilon}, \dot{\bar{\epsilon}}, T) \quad (3-15)$$

where $\bar{\epsilon}$ is the natural strain, $\dot{\bar{\epsilon}}$ is the strain rate, and T is the material temperature. Strain and strain rate can be defined as:

$$\bar{\epsilon} = \ln \frac{A_C}{A_E} \quad (3-16)$$

$$\dot{\bar{\epsilon}} = \frac{d\bar{\epsilon}}{dt} \quad (3-17)$$

where A_C is the cross-sectional area of the container, and A_E is the cross-sectional area of the extruded material.

Frictional pressure (P_F) is also a function of several parameters and can be expressed in functional form as (Saha 2000):

$$P_F = f(p_r, m, m', m'', D, L, L') \quad (3-18)$$

where p_r is the radial pressure, m is the friction factor between the billet and container wall, m' is the friction factor at the dead-material zone/flowing material interface, m'' is the factor of friction between die bearing and extruded material, D is the diameter of billet, L is the billet length, and L' is the die bearing length of a solid die.

Redundant pressure required for internal deformation work is a function of flow stress ($\bar{\sigma}$) and angle of dead material zone (α). Figure 3-15 shows different elements involved in redundant work for material deformation in the extrusion method.

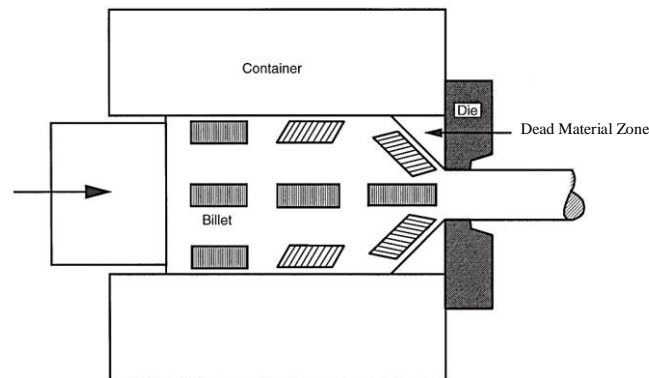


Figure 3-15: Elements involved in redundant work (Saha 2000)

In this figure, the elements around the perimeter of billet and elements at the interface of the dead-material zone are under huge shear deformation and elements at the centre of the billet is under an elongation in the extruded rod due to change in cross sections. Energy expenditure is required for shear deformation of material in extrusion process which is called redundant work (Saha 2000). Considering the meaning of work in physics, which is calculated by multiplication of the force by the amount of movement of the material, redundant work is the force required for deformation of material and movement of material.

3.2.3.6 Factors affecting extrusion pressure

Many factors are considered to choose proper equipment for having desired industrial products. Technological restrictions and economical aspects are among the most important factors for designing industrial extrusion equipment. If the length of billet increases, the frictional resistance increases which requires a more powerful press. By increasing the temperature of billet, the pressure required for deformation of material decreases, but it might have a negative effect on the extrusion equipment and final product. For a given extrusion equipment and billet with specific geometry and properties, there will be a maximum possible extrusion ratio. Even speed of extrusion should be controlled based on capabilities of equipment and quality of final products. All of these interrelated factors affect the required extrusion pressure. Generally, proper extrusion pressure for deformation of material in industrial extrusion process is decided based on practical restrictions and past experience (Saha 2000).

Whyte (1982) discusses a term of “short billet” length that is related to a ratio of the billet length to the diameter of the container (L / D) when this ratio is less than about 0.7 as it

is shown in diagrams of Figure 3-16. At this point, the punch load starts to increase in both direct and indirect methods by continuing ram travel. This ratio should not be an axiom in extrusion process as Kayabali and Tufenkci (2010a) did not observe any significant difference between the results obtained by different L / D ratios. Actually, it can be related to start of deformation of the dead material zone. The parameters affecting generation of this zone have been discussed earlier and are not related to billet length and diameter of container alone. For instance, by considering a direct extrusion device with the extrusion ratio of one, the punch load should decrease constantly after the start of extrusion by decreasing in container-billet friction. Hence, L / D ratio should not be an axiom in extrusion process.

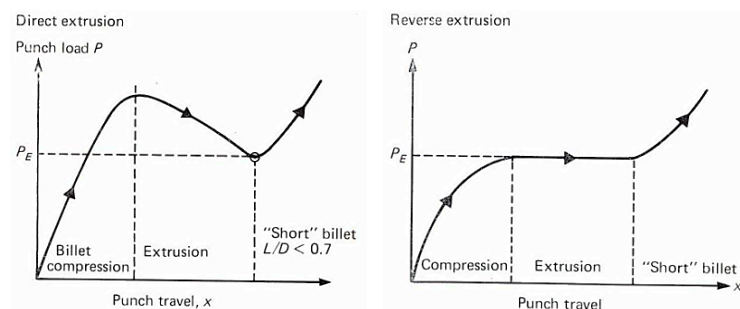


Figure 3-16: Typical extrusion punch loads in direct and reverse extrusion apparatus (Whyte 1982)

3.2.3.7 Extrusion pressure analysis

There are several theoretical analysis methods for determination of the average extrusion pressure during extrusion process. Thomsen et al. (1965) had combined uniform energy method, slab analysis, and slip-line field theory for extrusion pressure analysis. Altan et al. (1983) had also used slab method analysis for determination of extrusion pressure considering following assumptions:

- A cylindrical billet is extruded through a flat die
- Extrusion shape is equivalent to a rod with diameter of DE
- Frictional shear stress at the dead-material zone/flowing material interface
- Frictional shear stress at the interface of billet and container

Figure 3-17 shows two shaded elements in an extrusion of a rod through a square die with the dead-material zone. Considering equilibrium of forces on these elements will lead to the stresses acting on the slabs that are shown in Figure 3-18.

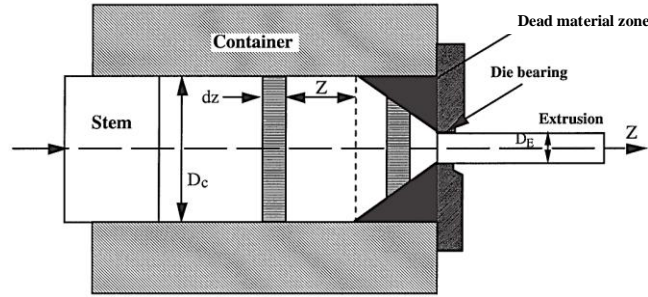


Figure 3-17: Extrusion of a rod through a square die with dead-material zone (Saha 2000)

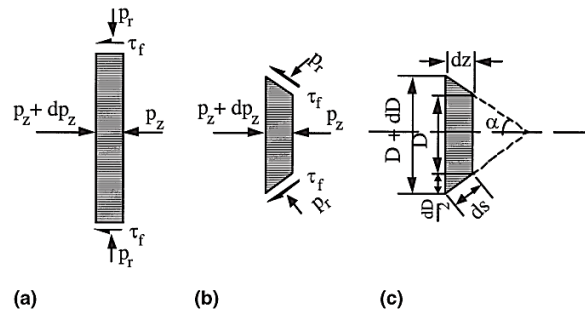


Figure 3-18: Stress states for the extrusion elements shown in Figure 3-17. (a) Free body diagram of element inside the container wall. (b) Free body diagram of element under the dead-material zone. (c) Geometric relationship between dz , dD , and ds (Saha 2000)

Considering Figure 3-18(b), the equilibrium equation is:

$$-(p_z + dp_z) \frac{\pi(D + dD)^2}{4} + p_z \frac{\pi D^2}{4} + p_r \pi D ds \sin \alpha + \tau_f \pi D ds \cos \alpha = 0 \quad (3-19)$$

which p_r is the radial pressure, τ_f is the frictional shear stress at the dead-material zone/flowing material interface, and α is the semidead-material zone angle.

Considering following geometric relationships among dz , dD , and ds :

$$ds \sin \alpha = dz \tan \alpha = \frac{dD}{2} \quad (3-20)$$

$$ds \cos \alpha = dz \cot \alpha = \frac{dD}{2 \tan \alpha} \quad (3-21)$$

Considering the yield criterion:

$$p_r = p_z + \bar{\sigma} \quad (3-22)$$

which p_r is the radial pressure, p_z is the pressure in the Z direction, and $\bar{\sigma}$ is the material flow stress.

The equilibrium equation can be obtained by combination of equations (3-19), (3-20), (3-21), (3-22), and substitution of τ_f from equation (3-13), and neglecting higher order differentials as:

$$\frac{dp_z}{\bar{\sigma}(1 + \frac{\cot \alpha}{\sqrt{3}})} = \frac{2dD}{D} \quad (3-23)$$

By assuming constant flow stress:

$$\frac{p_z}{\bar{\sigma}(1 + \frac{\cot \alpha}{\sqrt{3}})} = \ln D^2 C \quad (3-24)$$

which C is the constant of integration and can be eliminated by substitution of the boundary conditions at $D = D_E$, $p_z = 0$:

$$C = \frac{1}{D_E^2} \quad (3-25)$$

which D_E is the equivalent diameter for extruded rod and can be calculated by equation (3-9).

The average extrusion pressure can be obtained by substitution of value of the C constant in equation (3-24) as:

$$p_{ave, Z=0} = 2\bar{\sigma}(1 + \frac{\cot \alpha}{\sqrt{3}}) \ln \frac{D_C}{D_E} \quad (3-26)$$

which $\bar{\sigma}$ is the material flow stress, α is the semidead-material zone angle, D_C is the equivalent diameter of the billet, and D_E is the equivalent diameter of extruded rod.

For calculation of total pressure required for extrusion it is required to consider billet-container interface friction too. The equation of static equilibrium can be written by considering the free body diagram of the element inside the container wall shown in Figure 3-18(a):

$$[(p_z + dp_z) - p_z] \frac{\pi D_C^2}{4} = \pi D_C \tau_f dz \quad (3-27)$$

which D_C is the equivalent diameter of the billet, τ_f is the friction force at the billet-container interface. Integral form of this equation can be written as:

$$\frac{dp_z}{\tau_f} = \frac{4}{D_C} dz \quad (3-28)$$

which τ_f is the friction force at the billet-container interface, and D_C is the equivalent diameter of the billet.

The average extrusion pressure can be expressed by considering the boundary conditions at $Z = 0$, $p_z = p_{ave, z=0}$ as:

$$p_z = \frac{4\tau_f Z}{D_C} + p_{ave, Z=0} \quad (3-29)$$

which τ_f is the friction force at the billet-container interface, and D_C is the equivalent diameter of the billet.

Considering equations (3-26) and (3-13) and substitution of $p_{ave, z=0}$ and τ_f in equation (3-29), the average extrusion pressure can be expressed as:

$$p_{ave} = 2\bar{\sigma} \left(1 + \frac{\cot \alpha}{\sqrt{3}} \right) \ln \frac{D_C}{D_E} + \frac{4\bar{\sigma}Z}{\sqrt{3}D_C} \quad (3-30)$$

where $\bar{\sigma}$ is the material flow stress, α is the semidead-material zone angle, D_C is the equivalent diameter of the billet, and D_E is the equivalent diameter of extruded rod.

3.2.3.8 Extrusion force

Determination of force term is necessary for deciding about capacity of extrusion press. As it was discussed about extrusion pressure, extrusion force is also depend on the friction condition at the die material interface and the billet container interface, flow stress of the material, the extrusion ratio, the speed of extrusion, and temperature of billet (Saha 2000). The required extrusion force (F_r) can be expressed as:

$$F_r = P_T A_C \quad (3-31)$$

where P_T is the total extrusion pressure, and A_C is the cross-sectional area of the container.

It is required that the force applied by the press (F_p) be more than the force required for extrusion (F_r). In metal industries, it is required to provide very high compression force for metal extrusions. Hence, it might be necessary to combine several hydraulic compressors in this regard as it is demonstrated in Figure 3-19.

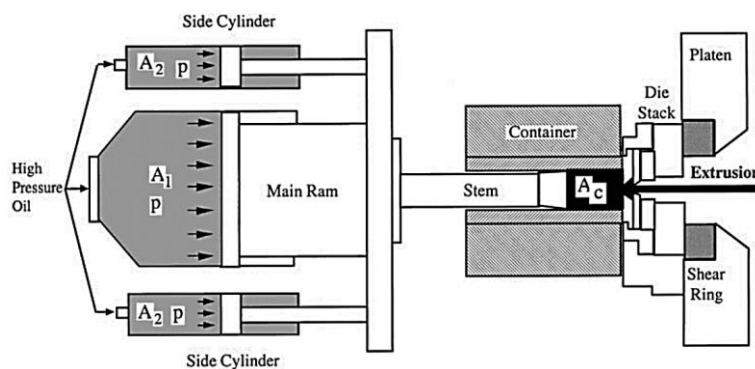


Figure 3-19: A schematic figure of providing compression power in metal industry (Saha 2000)

The compression power or the force applied by the press shown in Figure 3-19 can be expressed by:

$$F_p = pA_1 + p(2A_2) \quad (3-32)$$

which p is the applied pressure to the cylinders, A_1 is the main cylinder area, A_2 is the area of each side cylinder.

Figure 3-20 demonstrates inner pressure (specific pressure) in the container liner of extrusion equipment that can be calculated by:

$$P_s = \frac{F_p}{A_C} \quad (3-33)$$

which F_p is the force applied by the press, and A_C is the cross-sectional area of container.

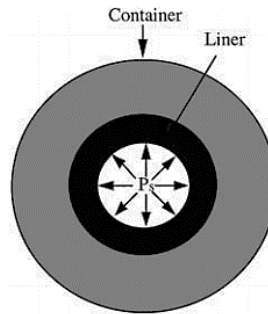


Figure 3-20: Inner pressure (specific pressure) (Saha 2000)

3.2.3.9 Effect of main parameters on extrusion

Industrial extrusion facilities are fairly expensive and it is required to control effective parameters on extrusion process based on the desired quality of the final product and economical aspects. As it was discussed, determination of the force required for compression of material is of great concern in industrial extrusion. The main variables influencing the required force in this regard is demonstrated in Figure 3-21 and are (Saha 2000):

- Extrusion ratio
- Extrusion temperature
- Extrusion speed
- Material flow stress

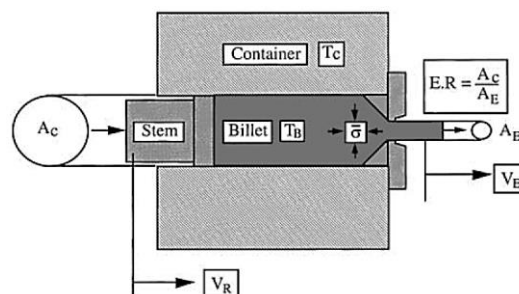


Figure 3-21: Main variables of industrial extrusion (Saha 2000)

Extrusion ratio

The extrusion ratio with several symmetrical dies can be defined by:

$$ER = \frac{A_C}{nA_E} \quad (3-34)$$

where A_C is the cross-sectional area of container, n is the number of symmetrical holes, and A_E is the cross-sectional area of extruded material.

The extrusion pressure is related to extrusion ratio indirectly considering that extrusion strain is a function of extrusion ratio. Plastic strain of material in extrusion process will be low if the extrusion ratio is low. Hence, the work required for deformation of material will be low and low extrusion pressure will be required for deformation of material (Saha 2000).

Products obtained by extrusion method with low extrusion ratio usually have similar mechanical properties of billet. However, it might not be desired in industrial products. For instance, in metal industry, it is planned to obtain a desired physical and mechanical properties through the extrusion process and it may not be guaranteed to reach to that propose with an extrusion ratio of less than 10 to 1 (Saha 2000). In contrast, if the extrusion ratio is high, higher amount of plastic strain will be happened and more extrusion pressure will be required. In general, the appropriate extrusion ratio would be obtained by practice. For instance in metal industry, the normal extrusion ratio for hard alloys is in range of 10/1 to 35/1, and for soft alloys is in range of 10/1 to 100/1 (Saha 2000). However, these are not the reference ratios but should be selected by experience.

Temperature of extrusion

The temperature in industrial extrusion is of great concern due to several reasons. In metal industry, since metals and alloys usually do not have desired plasticity range at room temperature and deformation of these materials through extrusion method requires extremely high pressure and special equipment, metal extrusion will be performed with a specific range of temperature to reduce the required extrusion force for metal formation (Saha 2000). In addition, recovery process requires specific temperature condition too. The temperature parameter is an important variable in industrial extrusion because the flow stress will be reduced by increasing the temperature of material and the material deformation will be easier. However, many factors should be considered in heating the material such as localized melting, heat transfer to equipment, and even heat developed by material deformation and friction (Saha 2000). Although temperature of soil billet in current study is not of great importance, increase in billet temperature in steel extrusion for reduction of extrusion pressure can be analogous with increase in water content in soil billet in current study.

Extrusion speed

The speed of deformation can affect the response of material in extrusion processes. Increase in extrusion pressure will lead to increase in ram speed and therefore increase in temperature developed during extrusion processes due to increase in strain rate (Saha 2000).

It is possible to calculate the extrusion speed by Utilisation of volume constancy relation between billet and extruded rod material during extrusion process. Based on this method and by considering volume constancy parameters shown in Figure 3-21, it can be written:

$$V_R A_C = V_E A_E \quad (3-35)$$

where V_R is the ram speed, A_C is the cross-sectional area of the container, V_E is the extrusion speed, and A_E is the cross-sectional area of extruded material. Hence, the extrusion speed can be expressed by:

$$V_E = V_R \frac{A_C}{A_E} = V_R ER \quad (3-36)$$

where V_R is the ram speed, A_C is the cross-sectional area of container, A_E is the cross-sectional area of extruded material, and ER is the extrusion ratio.

Material Flow Stress

The stress required for plastic deformation of material can be obtained from stress-strain curve (flow curve). The flow stress is affected by several factors (Saha 2000):

- Chemistry of material
- Structure of material
- Temperature of material
- Amount of deformation or strain
- Rate of deformation or strain-rate

Since flow stress is greatly affected by strain rate and temperature of material, there is no specific method for determining the flow stress in industrial metal extrusion. It is usually determined experimentally by common tests such as tensile, uniform compression, and torsion tests (Saha 2000).

3.3 Previous soil extrusion apparatus

As it was discussed earlier, many extrusion devices have been used for industrial purpose in different fields, but very few researches has been done for determination of soil properties as it was argued in chapter 2. Whyte and Kayabali are two recognised researchers in geotechnical engineering that have utilized extrusion method for determination of Atterberg limits (Whyte 1982; Medhat and Whyte 1986; Kayabali and Tufenkci 2007; Kayabali and Tufenkci 2010a; Kayabali and Tufenkci 2010b; Kayabali

2011a; Kayabali 2011b; Kayabali 2012b; Kayabali 2012a; Kayabali 2013; Kayabali and Ozdemir 2013; Kayabali et al. 2015a; Kayabali et al. 2015b; Kayabali et al. 2016). The general information about their method and the approaches they had used are discussed in chapter 2, but here the differences between their studies with the proposed method and properties of their apparatus will be discussed.

3.3.1 Previous soil extrusion approaches

As it was discussed in chapter 2, one of the popular approaches for determination of soil plasticity was correlating particular shear strengths to soils liquid and plastic limits. Early studies of soil extrusion were based on this wrong assumption which is strength based approach. In this approach, it was decided to correlate a specific extrusion pressure to a specific shear strength of soil and correlate it to Atterberg limits (Timar 1974; Whyte 1982; Medhat and Whyte 1986).

Another approach was to correlate the extrusion pressure obtained by a specific ram speed to the results obtained by current standard methods, without considering the intrinsic and reproducibility problems of current standard methods (Kayabali and Tufenkci 2007; Kayabali and Tufenkci 2010a; Kayabali and Tufenkci 2010b; Kayabali 2011a; Kayabali 2012b; Kayabali and Ozdemir 2013).

Alternative approach was a prediction of Atterberg limits by a multiple regression analysis on several extrusion pressures within plastic range of soil and the results of standard Atterberg limit tests, again without considering the intrinsic and reproducibility problems of current standard methods (Kayabali et al. 2015a; Kayabali et al. 2015b; Kayabali et al. 2016). It should also be noted that the procedure they had proposed is a prediction of liquid and plastic limits obtained by conventional methods and not a direct determination of these limits.

All of the proposed approaches utilising extrusion method have a common feature that they are following quantitative approach. In this approach, a parameter of extrusion process (extrusion pressure) with an arbitrary ram speed is measured, and it is tried to correlate it to Atterberg limits. As it was discussed in chapter 2, it is not appropriate to determine one particular property (i.e. extrusion pressure) that might cover some of the parameters or a portion of some parameters in determination of soil plasticity.

Author's suggestion for solving the problems related to determination of soil plasticity by proposed methods is utilisation of qualitative research. The reason is that the behaviour of soil in different states is a qualitative phenomenon which is related to many parameters that form the overall behaviour of a soil when its water content varies. Hence, it is not appropriate to determine one particular property. In contrast, the approach of the current study is the determination of overall behaviour of soil utilising extrusion technique. In this

technique, it is possible to determine the workability of soil at Atterberg limits involving affecting parameters on soil behaviour and specifying the consistency state of soil as it was explained earlier.

3.3.2 Specifications of previous soil extrusion apparatus

There is very few information about the apparatus themselves and the reasons for choosing different specifications of the apparatus the researchers had used for their investigation. Whyte (1982) provided an empirical relationship between the extrusion pressure, remoulded shear strength, container and die orifice diameters for a plastic clay. He chose the extrusion ratio of 40 (38 mm and 6 mm for container and orifice diameters respectively) without providing more details about the apparatus and the reasons for choosing that extrusion ratio. However, he had stated: "If the extruded soil worm is chosen to have a diameter of 6 mm, then a quick 'rolling bead' test can always be carried out as a check on the results if required (see BS 1377, 1975 in which a 6 mm bead is specified)". Maybe the reason of not providing detailed specifications of apparatus was that he had wanted to commercialize his method with further investigations as he stated: "A more accurate extrusion system developed for commercial and this will ..." and "It is be to emphasised that the extrusion data presented are of a preliminary nature that have to be confirmed by future tests on test equipment currently being developed". However, author of current research could not find any more paper from Whyte about soil extrusion except his collaboration with Medhat that part of their paper was related to soil extrusion and there was no more information about the apparatus itself (Medhat and Whyte 1986). Kayabali has worked on soil extrusion by two different apparatus utilising indirect and direct extrusion methods (Kayabali and Tufenkci 2007; Kayabali and Tufenkci 2010a; Kayabali and Tufenkci 2010b; Kayabali 2011a; Kayabali 2011b; Kayabali 2012b; Kayabali 2012a; Kayabali 2013; Kayabali and Ozdemir 2013; Kayabali et al. 2015a; Kayabali et al. 2015b; Kayabali et al. 2016). He and his colleagues have been working on an indirect extrusion apparatus from 2007 to 2015 consist of a load frame with a 10 kN load cell by a constant speed loading ram, a steel container with inner diameter of 38 mm and with an orifice of 6 mm (ER = 40) as it had been chosen by Whyte. The height of container was 150 mm, and the clearance between ram and container was 0.25 mm. They had tried to fabricate an apparatus as Whyte (1982) had been built and they did not provide any specific reasons for the dimensions and specifications of their apparatus. The apparatus they used for indirect extrusion method is shown in Figure 2-14.



Figure 3-22: Extract from Kayabali and Tufenkci (2010a). Kayabali and Tufenkci's apparatus components

In late 2015, Kayabali and his colleagues introduced a new direct extrusion apparatus with several orifices (Kayabali et al. 2015a; Kayabali et al. 2016). They named it Mud Press Machine (MPM) that consisted of a load-cell, a loading piston, and a 30 mm diameter container with 30 mm height and 28 holes of 2.5 mm diameter. A picture of their device is shown Figure 3-23.



Figure 3-23: Mud press machine (Kayabali et al. 2016)

They did not provide any reason for shifting from indirect extrusion method to direct extrusion method. They considered the amount of soil samples that might be obtained from field tests such as the standard penetration test and chose the dimensions of the container, but they did not provide any reason for the number and dimension of extrusion holes of the container. Maybe they have chosen an arbitrary extrusion ratio ($ER = \frac{A_C}{nA_E} = 5.14$; $ER^{-1} \approx 20\%$) and decided about the numbers, diameter and arrangement of holes by considering manufacture convenience too.

Their device consists of a display screen for reading the extrusion force that the operator should keep the extrusion force steady manually by using the loading arm watching the display screen. Hence, operator's performance might affect the results obtained by this method.

3.4 Standardising the new test methods

The proposed test method is an extrusion test that provides a partially confined pressure for soil specimen in a mould and lets the soil get deformed and pass through several

holes at the base of the container. This method is able to measure the workability of cohesive soils when their moisture content varies and correlates a specific workability to a specific consistency state of cohesive soils.

Large data from the experiments is required to be analysed to draw a conclusion, together with observations to verify the proposed method. The experiments should be designed in a way to investigate the effect of important parameters in soil deformation on the performance of the proposed method.

3.4.1 Calibration of apparatus

The purpose of this research is determination of soil plasticity by determination of liquid and plastic limits. In this regard, a new test method has been proposed. It is required to verify the results obtained by this method. There are two different approaches in this regard. The first one is based on the meaning of soil plasticity. In other words, the behaviour of soil in the limit states should be defined in the literature precisely. Accordingly, it would be possible to determine the water content of soil at the limit states exactly. Unfortunately, there is not a clear and precise definition of soil plasticity in the literature as it was discussed in chapter 2. Consequently, it is not possible to calibrate the new device based on the first approach.

The other approach is based on the results obtained by current accepted conventional methods. In this approach, it is required to correlate the results of new apparatus to the results obtained by current standard methods. In other words, it is tried to find the same results of current standard methods by applying the new proposed method. This approach would be appropriate in condition that the results of conventional methods are accurate and decisive. However, this condition does not exist in the case of determination of soil plasticity as it is discussed in Chapter 2.

Unfortunately, there are no reference soils that can be referred to for calibration of new proposed devices due to variability of soil properties in different soil types. Another applicable approach for calibrating new apparatus is the combination of these two approaches. In this approach, it will be assumed that although current standard methods do not specify the exact limit states of plastic soils, they can specify the limit states in a way that falls within a range of water content around real limit states. The margins of each limit state can be verified by doing several tests on various soil types and the best point of each domain, which the soils have the same soil behaviour in comparison to other soils, can be considered as the actual limit state. Accordingly, the meaning of each limit state can be redefined based on the concepts of the new proposed method by a back analysis method.

In this research, it will be tried to calibrate the new apparatus based on the third proposed approach. Therefore, the standard liquid and plastic limits of soil specimens will be determined according to current standard methods. Then, the behaviour of soils will be investigated at water contents around the limit states. Since the standard methods have their own deficiencies, which were discussed in chapter 2, the water content of soil at the standard limit state has a small difference with the water content of soil at real limit state. Thereafter, the acceptable boundaries of each limit state will be identified and the apparatus will be calibrated by the behaviour of the soils at the average of liquidity index of soils which is assumed to be the benchmark for calculation of real limit states. The liquidity index of soils is considered because the LIs of different soils at limit states are always constant ($LI = 0$ for PL , and $LI = 1$ for LL) but the water contents might be different. Subsequently, an appropriate definition for each limit state will be provided based on the workability of soil in the specified system.

3.4.2 Soil material selection and sample preparation

Applicable soil types for Atterberg limit tests are: clays, silts, clayey and silty sands (SC and SM), and some organic soils (FHWA-IF-02-034 2002). It is impractical to do the experiment for all types of soils due to time and budget restrictions of current research project. Hence, the experiment is limited to be done on some mineral soils (obtained from different soil samples such as clays, silts, clayey and silty sands, and artificially prepared soil samples in lab). However, the results should be useful for other types of cohesive soils such as organic and peat soils that is required to be investigated in further studies.

It would be a good option to work on natural soils coming from known sites with the desired soil types. Considering the large number of test required for the investigation and the time and cost of providing natural soil samples, another much practical option is to prepare soil samples in laboratory artificially by mixing different proportions of various soil types available in the laboratory. The range of liquid limit and plastic limit should be kept as wide as possible. Since it is difficult to obtain many natural soil samples covering a large range of plasticity, different proportions of clay, silt and sand will be mixed together. It is also possible to add commercial bentonite to the soil samples at certain increments to obtain soil samples with higher plasticity range. Since the purpose of this study is determination of consistency of cohesive soils, the source of soil samples is not of great importance. In other words, the soil specimen prepared in every method has a specific consistency that the proposed method or any other method should be able to determine that specific consistency. Although sample preparation is of great concern for providing accurate data for site characterization, this research project is not about

providing data for a particular geotechnical project. Hence, it is possible to prepare soil samples in laboratory and do not follow the regular procedures for obtaining samples for common site investigations. However, after preparation of samples, it is required to follow the standard methods of specimen preparation to have a reliable comparison between the results of proposed method and results obtained by conventional methods. Therefore, the soil samples are air-dried and then pulverised for sieving. Soil samples for the investigation will be prepared in the laboratory based on the requirements of the project and with varying proportions of clay, silt, sand, and commercial bentonite as it is demonstrated in Figure 3-24. In addition, considering the requirements of the research project, it is possible to prepare natural samples based on dry preparation method (ASTM D421-85 2002) or wet preparation method (BS 1377: Part 2: 1990: 4.2.4).

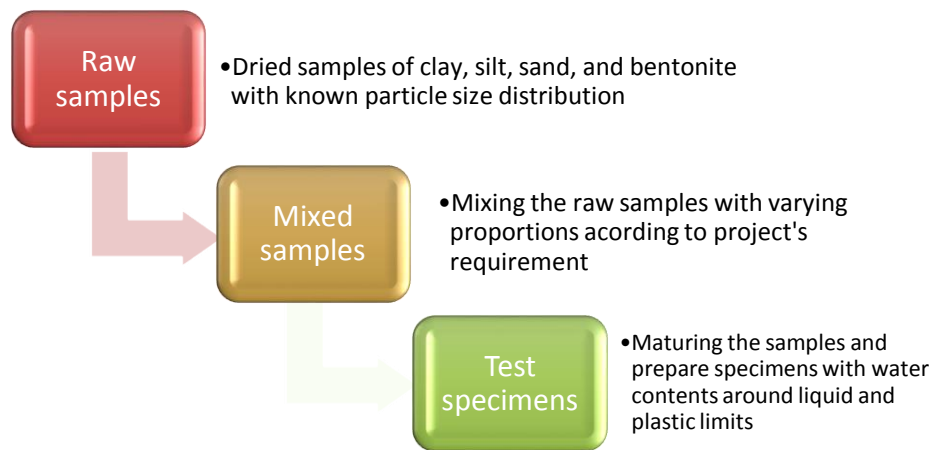


Figure 3-24: Artificial soil sample preparation

3.4.3 Laboratory tests

The soil tests that are done in this study can be categorised into two main groups:

3. Soil identification tests
4. Soil index tests

The soil identification tests are for recognition of physical properties of soil specimens under investigations. Since the tests are done on soil samples with less than 425-micron particle size, determination of particle size distribution are done using sieving and sedimentation test by hydrometer method according to BS EN 1377-2 (1990). For this purpose, particle density of soil samples will be determined by small pycnometer method according to BS EN 1377-2 (1990).

The soil index tests for determination of liquid and plastic limits of soil will include fall cone and thread rolling method according to BS EN 1377-2 (1990) in addition to proposed test method. The reason for choosing fall cone method is the advantages of this method in comparison to Casagrande percussion cup method, in addition to the

preference of this method in BS EN 1377-2 (1990) and Eurocode 7 (CEN. E.N. 1997-2 2007). However, both of standard tests should be considered for determining the acceptable domain of the liquid limit state. This can be done by considering data provided in literature as it will be discussed later.

3.5 Discussion

The main concept of proposed method for determination of soil consistency limits in this research project is the determination of overall behaviour of cohesive soil when its water content varies. Considering the concept of soil consistency discussed in chapter 2, it is proposed that soils should have the same behaviour at the liquid and plastic limits. Although soils might have different parameter values in various states, they should have the same overall behaviour at those specific limit states. The only variable that should be determined for having the same behaviour at different limit states is water content. It means that different soil types with different water contents should have the same behaviour at specific soil limit state. The main question is that: what system is appropriate for this purpose? For answering this question, it is required to consider the following points:

- The system should provide a specific situation for soil deformation that the behaviour of soil could be observed.
- Just change in moisture content of soil should affect the overall behaviour of soil in the system.

In other words, the lowest external factors (factors that are not pertinent to soil itself) affecting soil deformation, the more accurate results will be obtained. Extrusion method seems a good method for providing a particular system for measuring soil deformation with different water contents in a rational way.

There are several important factors for having a suitable final product in industries. These factors are interrelated and it is normally very hard to identify the contribution of each factor theoretically in the extrusion of material. In addition, economical aspects and technological restrictions should also be considered. Hence, designation of equipment for extrusion of industrial materials are highly dependent on past experience. Fortunately, the purpose of application of the extrusion method in this study does not have many of restrictions of industrial metal formation by the extrusion method. The main purpose of current study is observation and measurement of soil deformation and there is no final products or specific characteristics for the extruded material. Moreover, deformation of soil paste does not require high pressures provided by special equipment.

The reason for selecting direct extrusion method was its simplicity in addition that in short billet lengths there will be little friction between container and billet. Therefore it can be

ignored due to the low contact surface and the low extrusion pressure. Hence, the stated disadvantage of direct extrusion method against reverse extrusion method for having more side wall friction can be ignored. However, there has been reported a lower extrusion pressure for direct extrusion method in comparison to reverse extrusion method (Medhat and Whyte 1986). This can be a subject of another research project which is not among objectives of the current research. It should be noted that any change in design plan of apparatus requires a complete research for calibration of apparatus to determine the soil consistency limits.

In metal forming industry, the billet is heated to form the metal with lower pressures. In other words, the billet with higher temperature will get deformed easier than the billet with lower temperature with the same pressure. This situation can be expected for soil extrusion in this study by varying water content. The soil billet with higher water content will get deformed easier (faster) than the soil billet with lower water content. In this study, it is tried to calibrate a proper soil deformation rate based on plastic and liquid limits of soils. In other words, it is tried to measure the workability of soil specimen by measuring the work required for deformation of soil. An idealized physical model of Manafi's apparatus is shown in Figure 3-25. In this model, F_E is the extrusion force, F_s is the side wall friction between soil material and container wall, W_s is the weight of soil specimen, and F_D is the force required for deformation of soil.

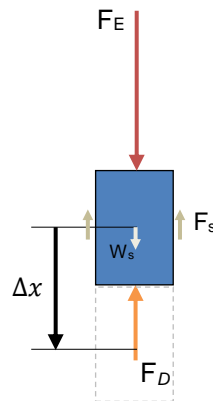


Figure 3-25: Idealised physical model of Manafi's apparatus

In the model, F_E is a constant force that should be great enough to overcome the F_D and F_s to be able to extrude the soil material. This force can be provided by a dead load on a load hanger acting on a rammer.

The required force for deformation of soil that will lead to extrusion of soil paste in the apparatus depends on several parameters that have been discussed in this chapter. The overall force required for overcoming resistant forces against deformation of soil in the apparatus is showed by F_D in this model. This force is high for soils with water contents around plastic limit and it is low for soils with water contents around the liquid limit.

As it was explained in “3.2.3.4 Friction models”, F_s can be assumed a small force for soils due to the low contact surface and low extrusion pressure in the container. However, the magnitude of F_s decreases continuously by extrusion progression as the contact surface decreases during the extrusion process. It can be assumed that this small force has not a significant effect on the overall deformation of soil in extrusion process assuming formation of pattern S or A in Figure 3-6 for soil paste extrusion. Another small force that acts during the extrusion process is the weight of soil specimen (W_s in Figure 3-25). Although different soils have different unit weight, the amount of gravitational force for most mineral soils in a small volume of container falls within a small range. Although the unit weight of organic soils can be different with mineral soils, the difference in a small volume is not significant. However, determination of plasticity of organic soil is not among the subjects of this research, but it can be a subject for further investigations in later research projects. In addition, like side wall friction, the amount of gravitational force acting on soil specimen decreases continuously while extrusion progresses and the soil extrudes through the holes. F_s and W_s are not considerable forces in comparison to F_E and F_D . Therefore, the two small opposing forces of W_s and F_s can be ignored during deformation of soil in the extrusion process in comparison to huge forces of F_E and F_D . However, the internal forces are considered as the F_D which will deform the soil inside the apparatus. Hence, the model would be simplified as it is shown in Figure 3-26.

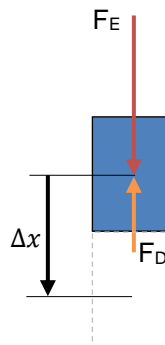


Figure 3-26: Simplified physical model of Manafi's apparatus

This model is a one-dimensional motion problem. The resultant force causes extrusion of soil ($F = F_E - F_D$). What is desirable in this study is F_D that works for deforming the soil inside the apparatus. Since a constant extrusion force (F_E) would be applied, it will be possible to find the F_D by determination of F . Hence, it would be possible to calculate the work of deformation force and by measuring its power, the workability of soil paste can be measured by following equations:

$$\bar{P}_D = \frac{W_D}{\Delta t} = \frac{F_D \Delta x}{\Delta t} \quad (3-37)$$

where W_D is the work done for deformation of soil, Δt is the time interval of soil deformation, F_D is the magnitude of force for soil deformation, and Δx is the magnitude of the soil specimen displacement.

$$F_D = F_E - F \quad (3-38)$$

where F_E is the magnitude of extrusion force, and F is the resultant force causing soil specimen acceleration.

$$F = ma \quad (3-39)$$

where m is the mass of soil specimen, and a is the acceleration of soil specimen.

$$\Delta x = v_0 t + \frac{1}{2} a t^2 = 0 + \frac{1}{2} a t^2 = \frac{1}{2} a t^2 \quad (3-40)$$

$$a = \frac{2\Delta x}{t^2}$$

where Δx is the magnitude of the soil specimen displacement, v_0 is the initial velocity of soil specimen at the start of extrusion which is zero, t is the time of extrusion, and a is the average acceleration of soil specimen.

It is possible to change the magnitude of deformation force by changing water content of soil specimen. Soil specimen with higher water content requires less power. Hence, it is possible to correlate a specific power (work / time) to a specific consistency state including liquid and plastic limits. It is tried to find the proper power for the proposed apparatus in this research. F_E and F_D are two variable forces that can be changed by varying dead load and extrusion ratio respectively. Changing these forces affects the power of apparatus and consequently the time of extrusion. For standardising the test method, it would be appropriate to find proper powers for liquid and plastic limits of having a fixed time of extrusion. Hence, it is tried to find the appropriate work by finding the proper F_E and F_D through trial and error. After calibration of the apparatus to a fixed power for each limit state, since the work is constant, it is just required to find the time of extrusion of soil specimen for several times with varying water contents to reach to the fixed time of extrusion. This can be achieved by plotting the water contents of soil specimens versus the time of extrusion and finding the water content pertinent to the fixed time which is related to the fixed workability of soil in that limit state as it is shown in Figure 3-27.

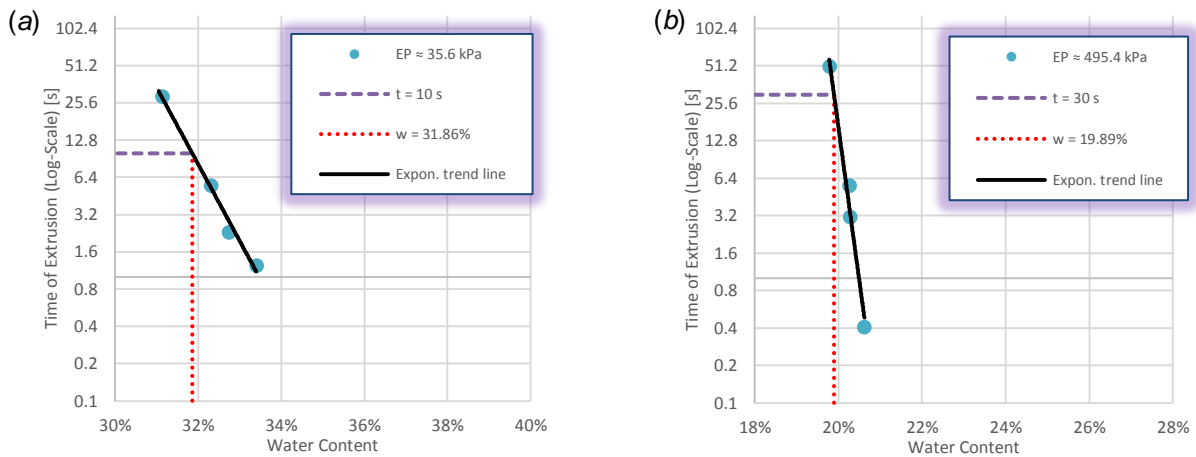


Figure 3-27: Determination of consistency limits by graphical method. (a) Determination of liquid limit. (b) Determination of plastic limit (Note: the power of apparatus is different in these limit states)

The new proposed method has several advantages to the conventional methods that make it a good candidate for replacement of current standard methods:

1. Appropriate research approach (qualitative approach) is followed in the test procedure for determination of qualitative phenomenon (soil plasticity)
2. Both of liquid limit and plastic limit tests are done by one apparatus and similar procedure.
3. Many of uncertainties of conventional methods expressed in Chapter 2 are eliminated in the proposed method.
4. It is not a time-consuming test.
5. The test requires a very low amount of soil sample.
6. Total volume of the soil specimen is tested unlike standard methods that a portion of the soil specimen is tested (i.e. soil around penetrated cone in cone penetrometer test). This makes the test more accurate in the case of any non-homogeneity in soil specimen.
7. This method is almost operator independent and can be done by inexperienced people with a quick introduction to the method.
8. The method is devised based on the concept of soil consistency discussed in chapter 2.
9. The apparatus is designed based on deformation of soil covering the most important factors affecting the behaviour of remoulded soils and not only one parameter such as soil undrained shear strength.
10. The apparatus is simple and lightweight that can be used anywhere even in field laboratories.
11. It is an inexpensive apparatus that can be manufactured easily.

The advantages of proposed method enable it to be patented as an alternative method for determination of liquid and plastic limits of cohesive soils.

3.6 Manafi's apparatus design evolution

The idea of designing a new apparatus for determination of soil plasticity came from a research topic for determination of Atterberg limits for peat soils following the research was done by O'Kelly (2016). The problems and ambiguities pertinent to Atterberg limits determination by conventional methods showed that these methods are not appropriate for evaluation of soil consistency for different soil types, especially for peats. Hence, it was decided to devise a new test method for determination of soils Atterberg limits without having problems of past methods. It has been always author's intention to determine overall behaviour of soils and correlate it to plastic and liquid limits. The first simple plans had some problems that led to some designs for covering those problems in the next plans in addition to providing operator's comfort. It was also necessary to design a simple and inexpensive apparatus that be efficient at the same time.

3.6.1 The first prototype of apparatus

It was decided to provide an under pressure system which in that, the effect of combination of various effective parameters on deformation of soil would be measured. Hence, it was designed to put the specimen under pressure in a mould and let the soil be extruded through two aperture at the bottom of the mould. For designing the apparatus, it was tried to design a simple apparatus to deform the soil. The details of the design plans are shown in Figure 3-28. The loading was provided by a simple load hanger, putting on top of extruder's stem. In this apparatus, the combination of various parameters which affect the deformation of soil could be calculated by measuring time duration of soil deformation in a specific defined system that has not the uncertainties and deficiencies of current standard methods for determination of soil plasticity.

Since it was intended to observe the capability of apparatus in deforming soils, it was decided to fabricate the first prototype of apparatus with the simplest form and by available materials in the laboratory to reduce the expenses as much as possible. The first prototype of Manafi's apparatus is shown in Figure 3-29.

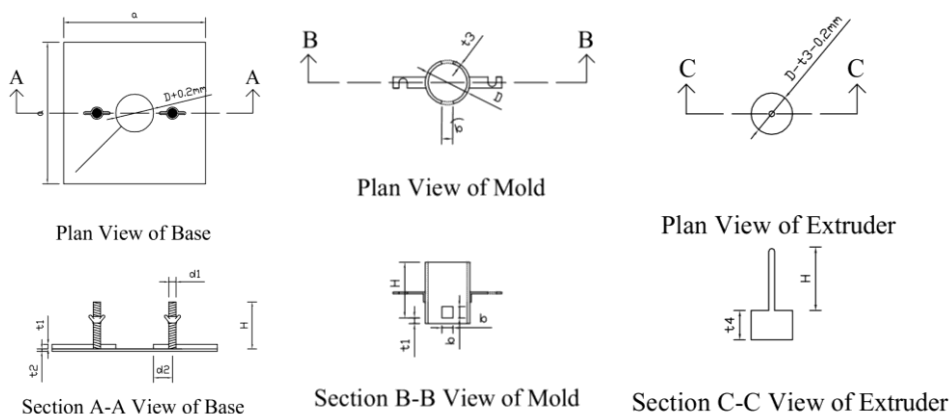


Figure 3-28: Details of design of the first prototype



Figure 3-29: The first prototype of Manafi's apparatus

Although the overall performance of apparatus showed its capability for deforming soil, fabrication with low-quality materials led to several problems that were necessary to be solved in next version of apparatus. The main problems were due to the usage of inappropriate materials and loading system which led to jamming of extruder inside the mould within first few centimetres of penetration.

3.6.2 The second prototype of apparatus

In this design, it had been tried to control vertical movement of the extruder by four linear shafts. In addition, a place for retrieving extruded soil has been designed that helps the operator for preparing soil paste with different water content for the next trial. The fabricated prototype is shown in Figure 3-30.

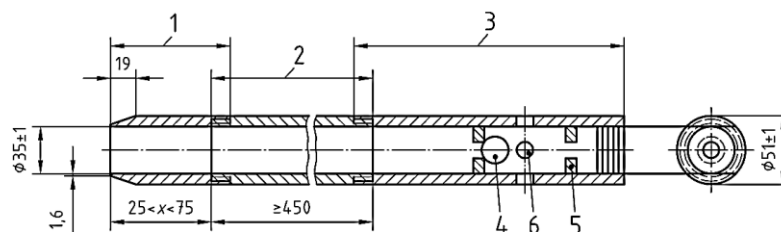


Figure 3-30: Manafi's second prototype

To make sure about the vertical movement of the rammer, four vertical guiding bars designed for base plate of apparatus that help the vertical movement of the extruder. It

was decided to fix four linear bearings in the holes of guider plates to reducing the friction between the bars and the guider plates as much as possible. A simple sliding gate enabled filling the container without wasting soil material with high water content. Dimensions of mould had chosen in way to contain about 300 ml of soil paste to provide enough space that maybe in later studies coarser particles of soil will get involve in determination of soil consistency. The diameters of the container and die orifice had been chosen in a way that the ratio of surface areas be about 10 for the first trial considering the effect of extrusion ratio on extruded material and extrusion force that is discussed in “3.2.3.90 Extrusion ratio”. This ratio chose for the first trial to be selected based on appropriate deformation rate and practical considerations.

Since it was a prototype, it was decided to fabricate the apparatus with available materials to get preliminary data to support the proposed method. It was decided to reduce the volume of the container. The big volume of the container in previous designs could enable the author for investigating the effect of coarser soil particles and maybe investigating consistency limits of soils with a full range of particle size in soil. However, having a smaller container has its own advantages. Smaller container requires less soil material to get filled which would be a great help in providing natural soils or artificial soil specimen built in the laboratory for many tests ahead. Preparation of soil specimen for next trials of changing water content and preparing a homogenous specimen is less cumbersome. The size of container chose the same as the size of a split barrel of standard penetration test (shown in Figure 3-31) that makes it possible to use unremoulded soil obtained by SPT in the same apparatus and have a comparison with the results obtained by remoulded soils. Although SPT cannot provide a perfect unremoulded soil specimen, the degree of remoulding is much less than remoulded soil specimens prepared for Atterberg tests. However, this investigation is beyond scope of current study.



Key

- 1 Drive shoe
- 2 Split barrel
- 3 Coupling
- 4 Non return valve (ball diameter: recommended 25 mm; ball seating: recommended 22 mm)
- 5 Ball retaining pins
- 6 Four vent holes (min diameter 12 mm)
- x Length of the drive shoe

Figure 3-31: Extract from BS EN ISO 22476-3:2005+A1 (2011). Cross section of an SPT sampler (dimensions in mm)

Since the linear bearings were not available in the workshop, it was decided to just bore 4 holes on guider plates without using linear bearings. It should be noted, as the guider plates move along the guide bars and there is no significant horizontal load during the extrusion process, there will not be much friction between the guider plates and the guide bars and it can be ignored in the performance of the apparatus prototype.

In addition, the design of the container die changed to multi-hole for reducing the effect of the dead-material zone on extrusion process (explained in “3.2.3.2 Dead-material zone”). The size of holes chose in a way that though the volume of the container does not let to investigate the behaviour of soil with a full range of soil particle size, all soil particles smaller than upper limit of sand size (4.75 mm and 2 mm based on ASTM and BS standards respectively) could pass through the die. Hence, 5 mm holes had decided for the prototype considering the fabrication practicality (using 5 mm drill bit). The number of holes had been decided based on an arbitrary extrusion ratio. At first, it had been decided to start from extrusion ratio of 10 and decrease it gradually to find the proper extrusion ratio for the purpose of research. But, due time and budget restrictions, it was decided to start from extrusion ratio of 3.27 ($ER^{-1} = 30.6\% \approx 30\%$) to reduce the force required for the extrusion process. Hence, 15 holes of 5 mm diameter that were distributed evenly in the base of the container with the diameter of 35 mm and height of 50 mm.

Since the container and the rammer are apart from each other, it is required to fabricate the elements with high precision equipment that are not available in the College's workshop. Although it was tried to fabricate it precisely, the adjustment of guider bars, guider plates, extruder, and container was difficult. It was the result of the combination of various clearance of different parts of apparatus between the guider bars and the base plate, the guider plates and the guider bars, the extruder and the guider plates, the container and the extruder, and the base plate and the container. Having a small gap between the guider plates and the guider bars for having a little friction had led to a little horizontal movement of the extruder that was detectable by shaking the extruder. Moreover, tight tying the nuts of the guide bars fixer plate led to high friction between the guider plates and the guider bars. It was due to slightly incompatible positions and distances of holes of the guider bars in the base plate and the guide bars fixer plate that led to a little inclination of the guide bars from the vertical direction. However, the friction between the guider plates and the guider bars highly decreased by tying the nuts a little loose. These led to having a gap of 0.6 mm between the rammer and the container wall. The data obtained by the second prototype were encouraging enough to cover the deficiencies in the next prototype.

3.6.3 The third prototype of apparatus

Considering the mechanical problems of the second prototype, a new design of apparatus covered the past deficiencies in addition to making the procedure of testing easier for an operator. For instance, the previous mould was heavy (1.2 kg) which made it cumbersome to wash, clean and dry the mould after each extrusion. In addition, it was required to design the apparatus to try various extrusion ratios. Various extrusion ratios tried and finally the chosen ER^{-1} for determination of LL and PL in the third prototype were 14.3% (7 apertures with a diameter of 5 mm) and 57.1% (7 apertures with a diameter of 10 mm) respectively. The reason for choosing these extrusion ratios was to have a comparison with the results of second prototype (about half and twice of 30.6%) in addition to fabrication practicality and operator's convenience. The fabricated prototype is shown in Figure 3-32.



Figure 3-32: Manafi's third prototype

4 Experimental Works and Data Analysis

4.1 Introduction

The experimental works were designed to determine Atterberg limits of soil by proposed and standard methods in addition to soil identification tests for determination of physical properties of soil samples. In this study, the last two prototypes were investigated extensively. It was tried to cover the problems of the second prototype in the third prototype. This chapter contains results together with data analysis for both liquid and plastic limits and some other consistency states investigated on ten different soils from various soil types for justification of proposed method: three natural soils (soils number 1, 2, and 4), one commercial soil (soil 3 – clay pellets), and six artificial soil samples by mixing different soils (soils number 5 to 10).

The soils were selected carefully to cover a wide range of soils plasticity. The plastic limits range of soils is from 18.64% to 30.78%, and the liquid limits range is from 30.25% to 61.77%. The wide range of soils plasticity shows that the selected cohesive soils are good samples to investigate for justification of proposed method.

4.2 Soil samples specifications

The soil identification tests are for determination of physical properties of soil specimens under investigations. Since the tests are done on soil samples with less than 425-micron particle size, determination of particle size distribution are done using sieving and sedimentation test by hydrometer method according to BS EN 1377-2 (1990). For this purpose, the particle density of soil samples will be determined by small pycnometer method according to BS EN 1377-2 (1990).

4.2.1 Particle density determination of soil samples

Particle density of soils is determined based on BS 1377-2:1990:8.3. The results are presented in Table 4-1.

Table 4-1: Particle density of soils according to BS 1377-2:1990:8.3

Soil Sample	1	2	3	4	5	6	7	8	9	10
Particle density (ρ_s) [Mg/m ³]	2.712	2.657	2.721	2.720	2.685	2.720	2.682	2.720	2.720	2.714

4.2.2 Particle size distribution (PSD) curves

Determination of particle size distribution is done using sieve analysis and sedimentation test by hydrometer method according to BS EN 1377-2 (1990). The results are shown in Figure 3-3.

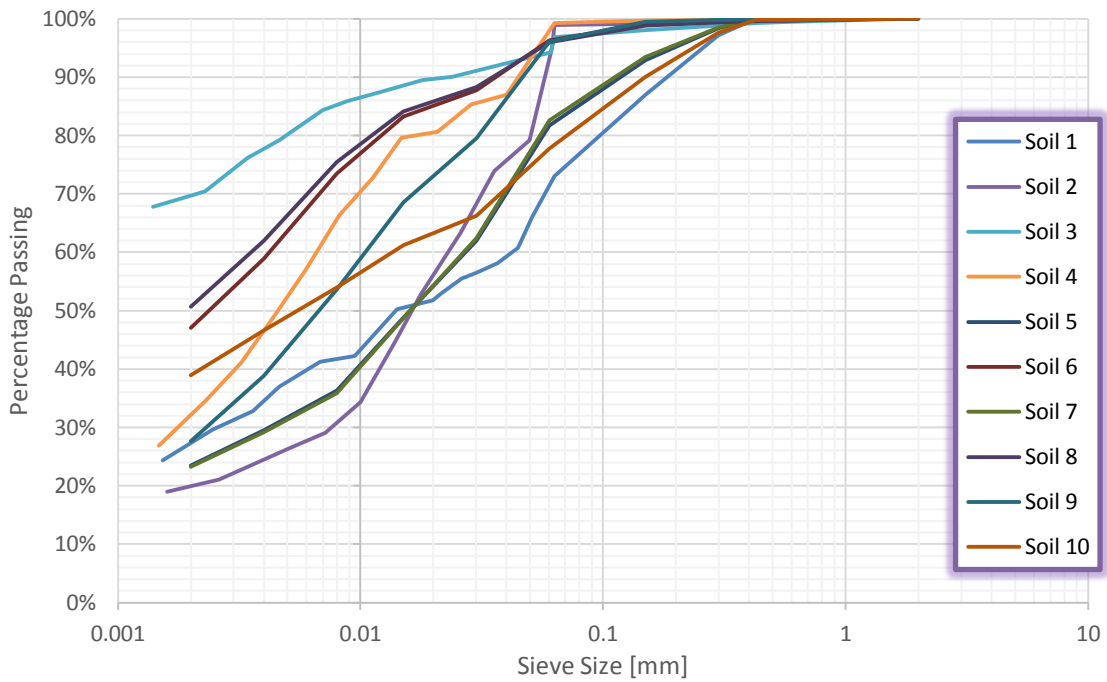


Figure 4-1: Particle size distribution curves

4.2.3 PSD analysis

To have a clear comparison among the soils based on soils PSD curves, each soil is subdivided into its soils components according to BS EN 1377-1 (1990): coarse sand, medium sand, fine sand; coarse silt, medium silt, fine silt; and clay, as shown in Figure 2-9. The results are presented in Table 4-2 and Figure 4-3.

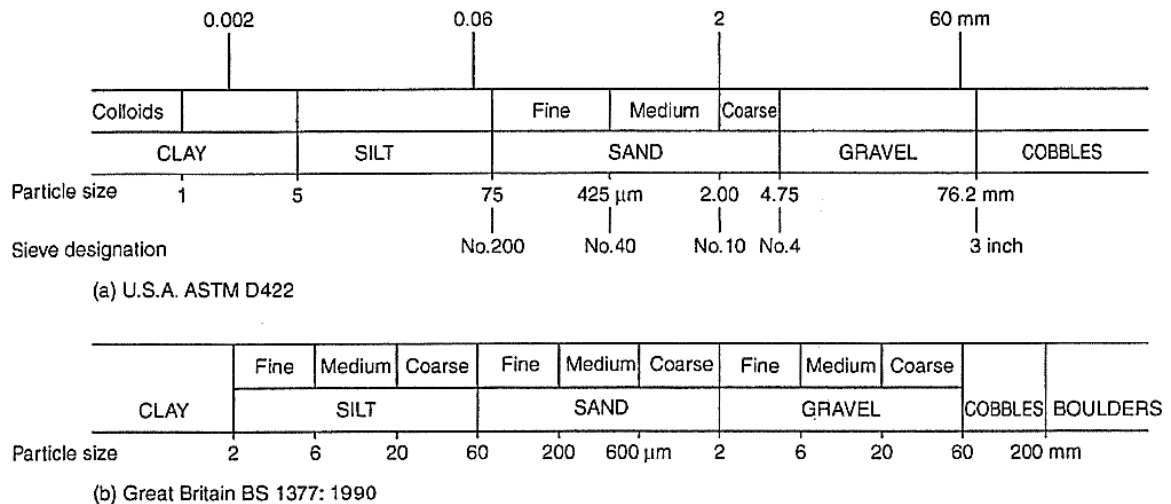


Figure 4-2: Extract from Head (2006). Classification of particle size ranges of soils

Table 4-2: PSD analysis based on BS 1377: 1990

Soil Components		Soil 1		Soil 2		Soil 3		Soil 4		Soil 5	
Sand	Coarse Sand	0.0%	28.7%	0.1%	7.3%	0.6%	5.9%	0.1%	2.2%	0.0%	18.3%
	Medium Sand	9.6%		0.6%		1.1%		0.2%		5.2%	
	Fine Sand	19.1%		6.7%		4.2%		1.9%		13.0%	
Silt	Coarse Silt	19.4%	44.3%	36.9%	72.9%	4.4%	24.6%	17.3%	66.1%	28.5%	58.2%
	Medium Silt	12.2%		28.2%		7.5%		23.3%		20.3%	
	Fine Silt	12.6%		7.7%		12.7%		25.5%		9.4%	
Clay	Clay	27.0%	27.0%	19.8%	19.8%	69.6%	69.6%	31.7%	31.7%	23.5%	23.5%

Soil Components		Soil 6		Soil 7		Soil 8		Soil 9		Soil 10	
Sand	Coarse Sand	0.3%	3.7%	0.0%	17.4%	0.3%	4.0%	0.3%	3.7%	0.2%	22.3%
	Medium Sand	0.6%		4.9%		0.7%		0.6%		7.2%	
	Fine Sand	2.8%		12.5%		3.0%		2.8%		14.9%	
Silt	Coarse Silt	11.6%	49.3%	29.3%	59.4%	10.5%	45.3%	11.6%	49.3%	14.9%	38.7%
	Medium Silt	18.5%		20.8%		16.8%		18.5%		12.6%	
	Fine Silt	19.2%		9.3%		18.0%		19.2%		11.3%	
Clay	Clay	47.0%	47.0%	23.2%	23.2%	50.6%	50.6%	47.0%	47.0%	39.0%	39.0%

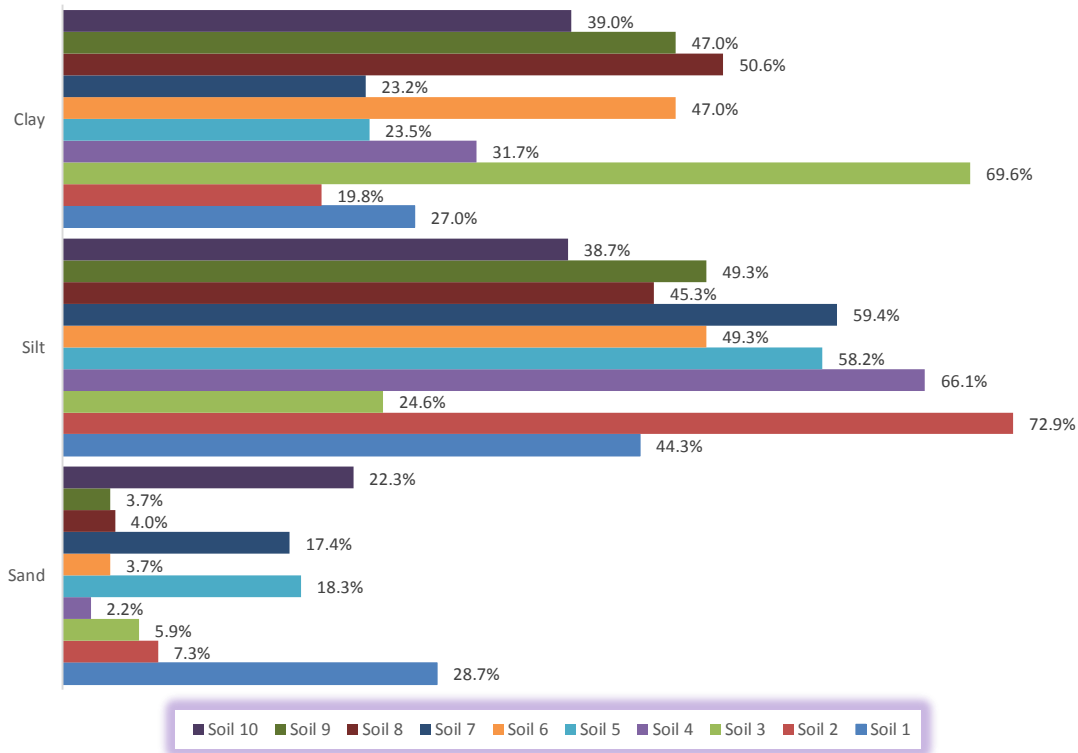


Figure 4-3: PSD comparison based on BS EN 1377-1 (1990)

PSD comparison shows that the soils cover a wide range of particle sizes with different proportions that make them good samples to investigate for justification of proposed method.

4.2.4 Plasticity determination by standard methods

Liquid and plastic limits of soils are determined by fall cone and thread rolling methods according to BS EN 1377-2 (1990). The results are presented in Table 4-3 and Figure 4-4.

Table 4-3: Atterberg limits by fall cone and thread rolling methods

Soil No.	Liquid Limit	Plastic Limit	Plasticity Index
1	33.86%	19.76%	14.10%
2	56.18%	30.78%	25.40%
3	61.77%	26.48%	35.29%
4	30.25%	20.11%	10.14%
5	44.49%	25.38%	19.11%
6	39.33%	20.87%	18.45%
7	45.24%	26.04%	19.20%
8	43.78%	20.20%	23.57%
9	38.08%	21.55%	16.54%
10	35.96%	18.64%	17.32%

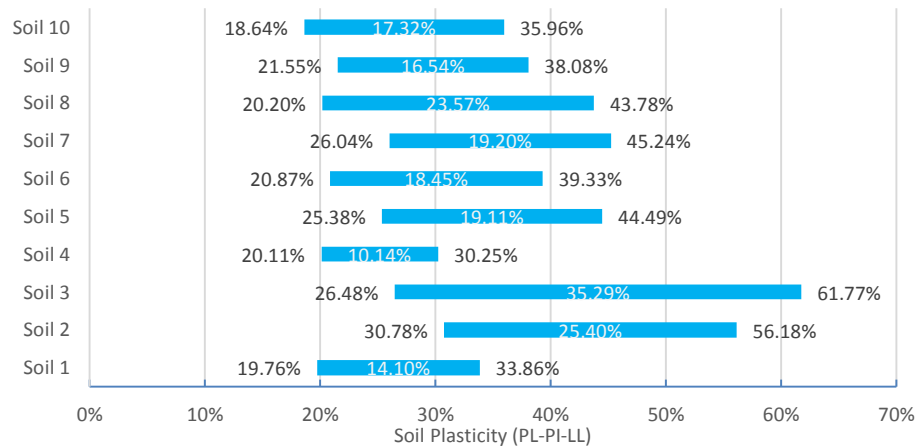


Figure 4-4: Soils plasticity comparison

The soils were selected carefully to cover a wide range of soils plasticity. The plastic limits range of soils is from 18.64% to 30.78%, and the liquid limits range is from 30.25% to 61.77%. To have a comparison with other similar researches, Sivakumar et al. (2015) have worked on 11 soils with plastic limits range of soils from 16% to 33.8%, and the liquid limits range from 36% to 77%. As it can be seen, there is not much difference between soil plasticity ranges covered in these researches. The wide range of soils plasticity shows that the selected cohesive soils are good samples to investigate.

4.3 Manafi's second prototype

As it was explained in chapter 3, the second prototype had a fixed extrusion ratio of 3.27 ($ER^{-1} = 30.6$). For calibrating the prototype, the extrusion pressures and extrusion time for determination of liquid and plastic limits have obtained by trial and error method. In this regard, several soil samples with water contents around the consistency limit states

were tested by different extrusion pressures and extrusion time to obtain the proper power for soil deformation in the prototype. Accordingly, extrusion pressures of 34.76 kPa and 635.98 kPa with extrusion time of 20 s obtained for determination of liquid and plastic limits respectively. The extrusion pressures of 143.99 kPa and 253.26 kPa were chosen arbitrarily to have two points within the plastic range of soils and study the capability of the prototype and the method for determination of any arbitrary consistency state of soils. Although the prototype had some minor mechanical problems that caused some data noise in the graphs, the results verified the proposed qualitative research method. The results by the second prototype encouraged the author to cover the problems in the next prototype.

4.3.1 Soil behaviour determination by the second prototype

After calibration of the prototype with several soils, all other soil samples were tested with the calibrated prototype. The soil behaviour determination tests by proposed method have been done on six soil samples. The results are presented in Table 4-4 to Table 4-9 and Figure 4-5 to Figure 4-10. By gathering data from proposed method and standard methods, it is possible to have a comparison between the proposed method and standard methods. For instance, Figure 4-5 (a) shows deformation rate of soil 1 with different water contents and compare them based on standard method liquidity indices. As explained in chapter 3, soils at their limit states should have the same deformation power (a calculation sample is provided in “4.5 Data analysis”). Figure 4-5 (a) shows the liquidity indices of soil 1 based on standard methods with the calibrated soil deformation rates. This graph shows that the water contents obtained by proposed method are very close to limit states obtained by standard methods which confirm the proposed method assumption (i.e. 1.03 and 0.09 for liquid and plastic limits respectively). Figure 4-5 (b) and (c) show the method of limit states determination by proposed method and compare the results with limit states obtained by standard methods.

Table 4-4: Soil behaviour determination tests on soil 1 by the second prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
90.94	34.76	34.20%	1.02	16.13	96.81	143.99	27.77%	0.57	3.84
91.92	34.76	35.04%	1.08	16.57	96.5	143.99	27.83%	0.57	2.00
91.49	34.76	35.14%	1.09	8.72	86.06	143.99	28.01%	0.58	11.25
92.24	34.76	35.69%	1.13	5.63	94.39	143.99	30.13%	0.74	0.35
91.37	34.76	35.81%	1.14	1.93	100.3	253.26	24.72%	0.35	22.43
91.54	34.76	35.99%	1.15	8.18	99.28	253.26	24.96%	0.37	4.32
92.31	34.76	36.33%	1.18	2.22	99.61	253.26	25.19%	0.39	2.47
91.39	34.76	36.51%	1.19	1.69	101.03	635.98	21.14%	0.10	20.36
96.55	143.99	26.78%	0.50	36.82	99.8	635.98	21.81%	0.15	2.41
94.52	143.99	27.50%	0.55	1.82	99.1	635.98	22.77%	0.21	0.47

* Decimal values of water content have been considered for more accuracy.

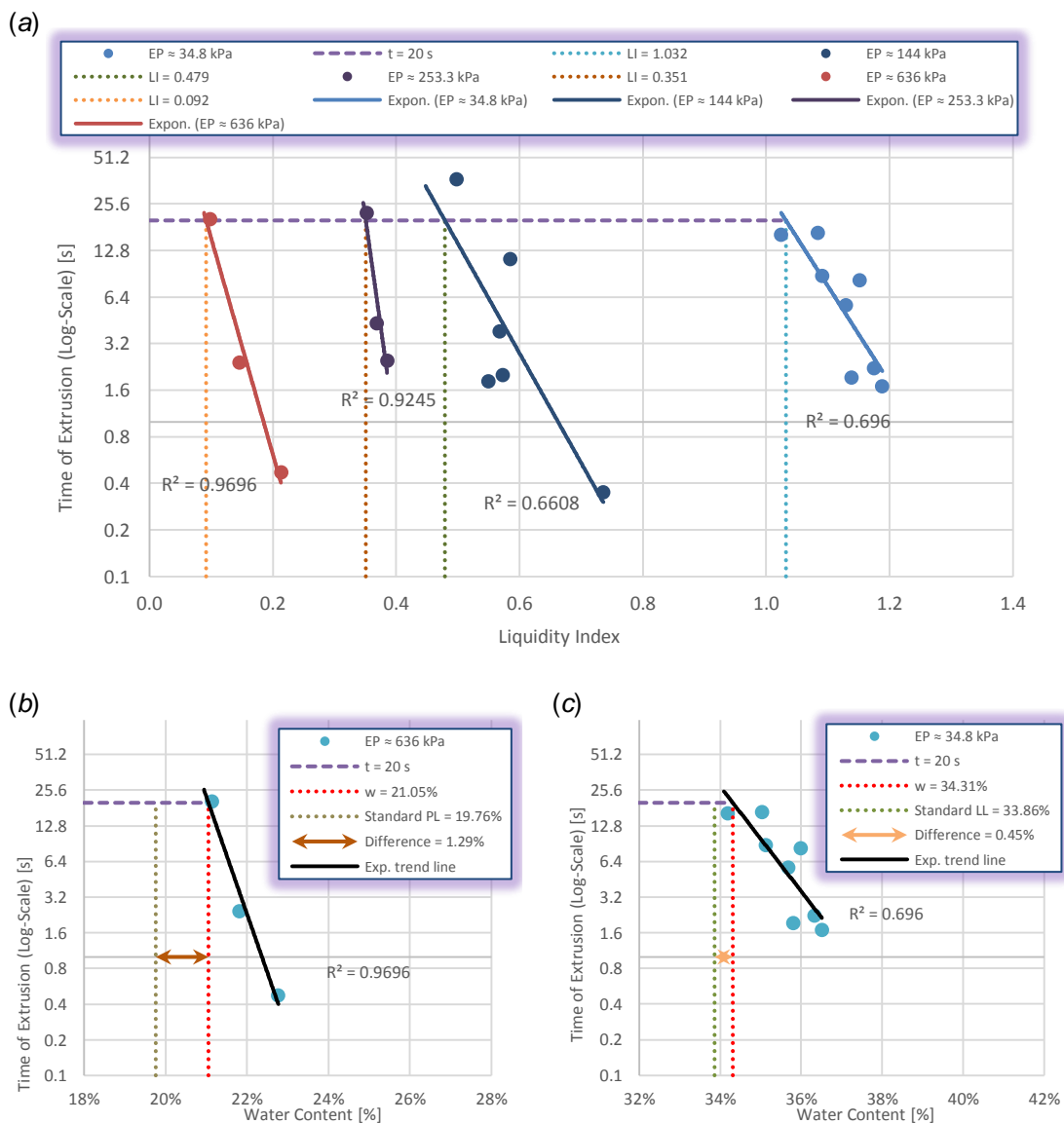


Figure 4-5: Soil 1 behaviour determination graphs by the second prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-5: Soil behaviour determination tests on soil 2 by the second prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
83.09	34.76	55.87%	0.99	12.72	82.08	143.99	43.34%	0.49	48.65
83.16	34.76	55.87%	0.99	11.79	89.99	253.26	38.17%	0.29	32.13
79.99	34.76	55.88%	0.99	29.40	90.55	253.26	38.41%	0.30	22.82
82.44	34.76	56.97%	1.03	4.79	91.34	253.26	38.57%	0.31	16.50
82.14	34.76	57.46%	1.05	13.06	89.49	253.26	38.72%	0.31	2.63
82.55	34.76	57.61%	1.06	6.28	89.93	253.26	38.77%	0.31	3.56
80.94	34.76	58.21%	1.08	5.41	89.63	253.26	39.33%	0.34	3.72
81.55	34.76	63.05%	1.27	0.63	89.69	253.26	39.83%	0.36	3.50
83.49	143.99	49.77%	0.75	0.69	87.03	253.26	41.54%	0.42	0.72
83.65	143.99	49.75%	0.75	0.53	88.27	253.26	41.99%	0.44	0.68
83.54	143.99	47.06%	0.64	1.00	93.06	635.98	29.80%	-0.04	2.50
85.22	143.99	45.53%	0.58	3.69	94.18	635.98	29.74%	-0.04	22.60
86.04	143.99	44.33%	0.53	7.53	94.27	635.98	29.54%	-0.05	111.81
86.39	143.99	43.74%	0.51	21.78					

* Decimal values of water content have been considered for more accuracy.

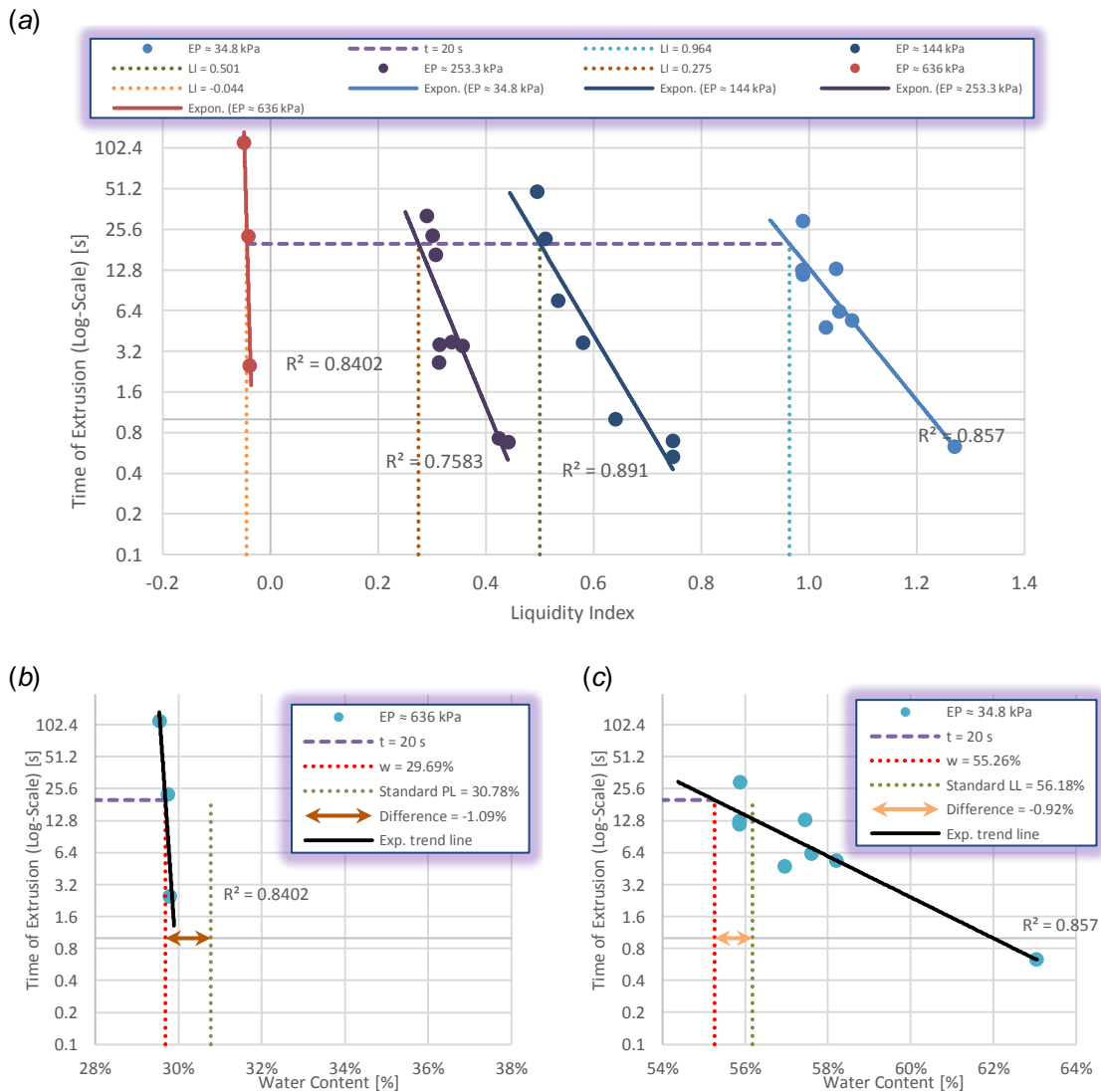


Figure 4-6: Soil 2 behaviour determination graphs by the second prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-6: Soil behaviour determination tests on soil 3 by the second prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
78.48	34.76	65.35%	1.10	0.56	89.51	253.26	38.76%	0.35	1.15
81.26	34.76	59.81%	0.94	2.75	90.53	253.26	37.92%	0.32	3.16
81.00	34.76	58.21%	0.90	23.75	91.31	253.26	36.55%	0.29	11.85
81.05	34.76	56.60%	0.85	98.66	92.18	253.26	35.34%	0.25	30.37
84.46	143.99	45.78%	0.55	0.46	95.03	635.93	30.86%	0.12	1.51
85.38	143.99	42.59%	0.46	2.50	95.1	635.93	30.28%	0.11	0.84
87.31	143.99	41.51%	0.43	8.03	95.58	635.93	29.35%	0.08	1.84
86.76	143.99	40.21%	0.39	18.50	95.02	635.93	28.32%	0.05	5.91
86.03	143.99	39.60%	0.37	39.72	96.47	635.93	27.63%	0.03	31.41

* Decimal values of water content have been considered for more accuracy.

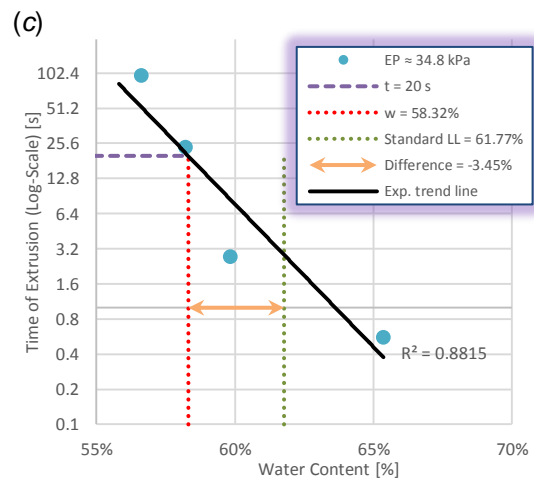
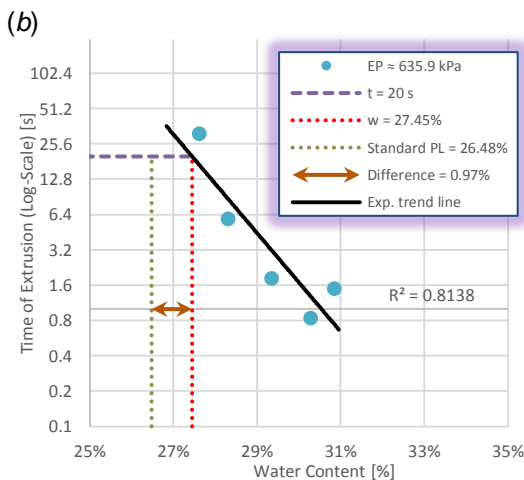
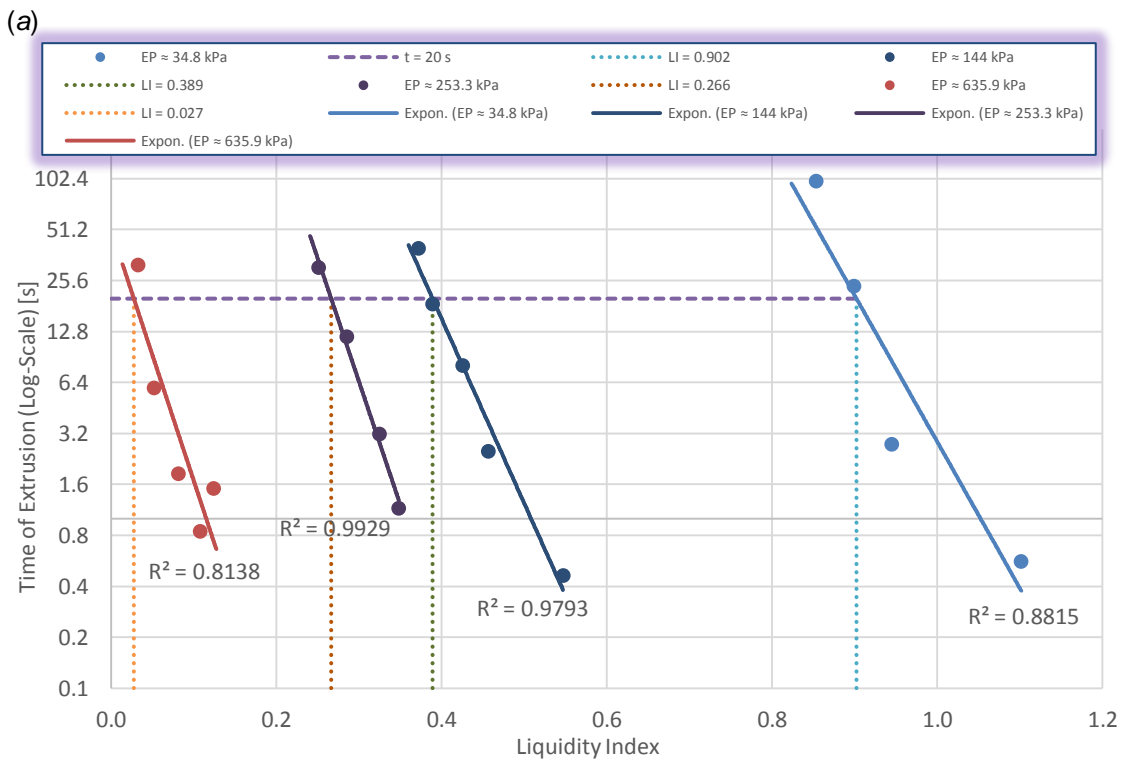


Figure 4-7: Soil 3 behaviour determination graphs by the second prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-7: Soil behaviour determination tests on soil 4 by the second prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
95.88	34.76	33.04%	1.28	2.57	96.86	34.76	31.92%	1.16	21.37
96.01	34.76	32.44%	1.22	6.30	103.91	635.98	21.87%	0.17	0.62
95.97	34.76	32.33%	1.20	7.78	104.6	635.98	20.08%	0.00	18.91
96.74	34.76	32.27%	1.20	11.22	104.16	635.98	19.69%	-0.04	106.28

* Decimal values of water content have been considered for more accuracy.

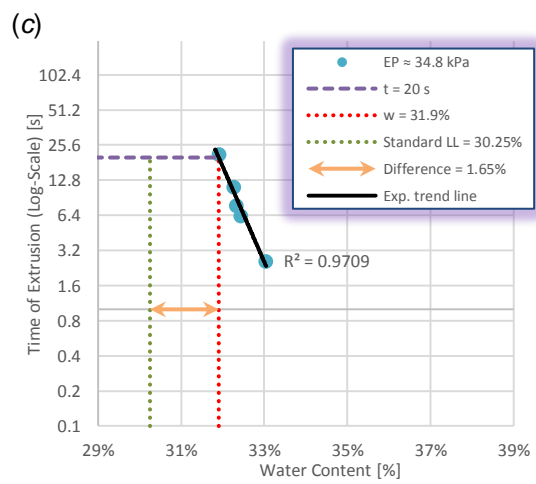
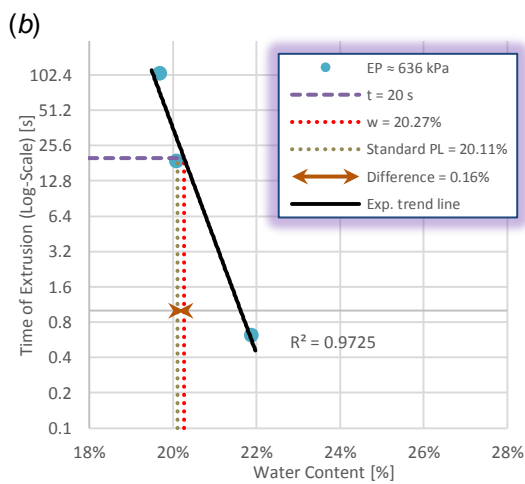
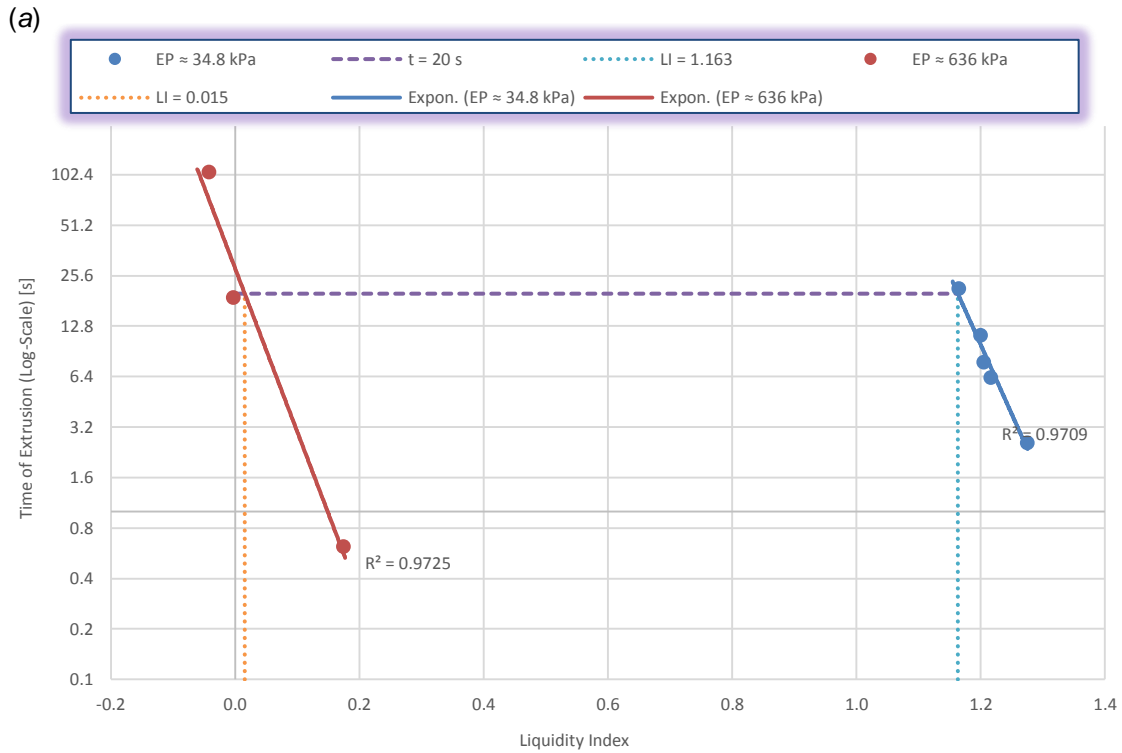


Figure 4-8: Soil 4 behaviour determination graphs by the second prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-8: Soil behaviour determination tests on soil 5 by the second prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
88.2	34.76	47.66%	1.17	0.86	95.97	635.98	28.39%	0.16	0.47
88.39	34.76	45.63%	1.06	2.69	96.42	635.98	26.45%	0.06	4.84
88.73	34.76	44.77%	1.01	7.25	97.92	635.98	25.44%	0.00	48.37
89.2	34.76	42.94%	0.92	283.00					

* Decimal values of water content have been considered for more accuracy.

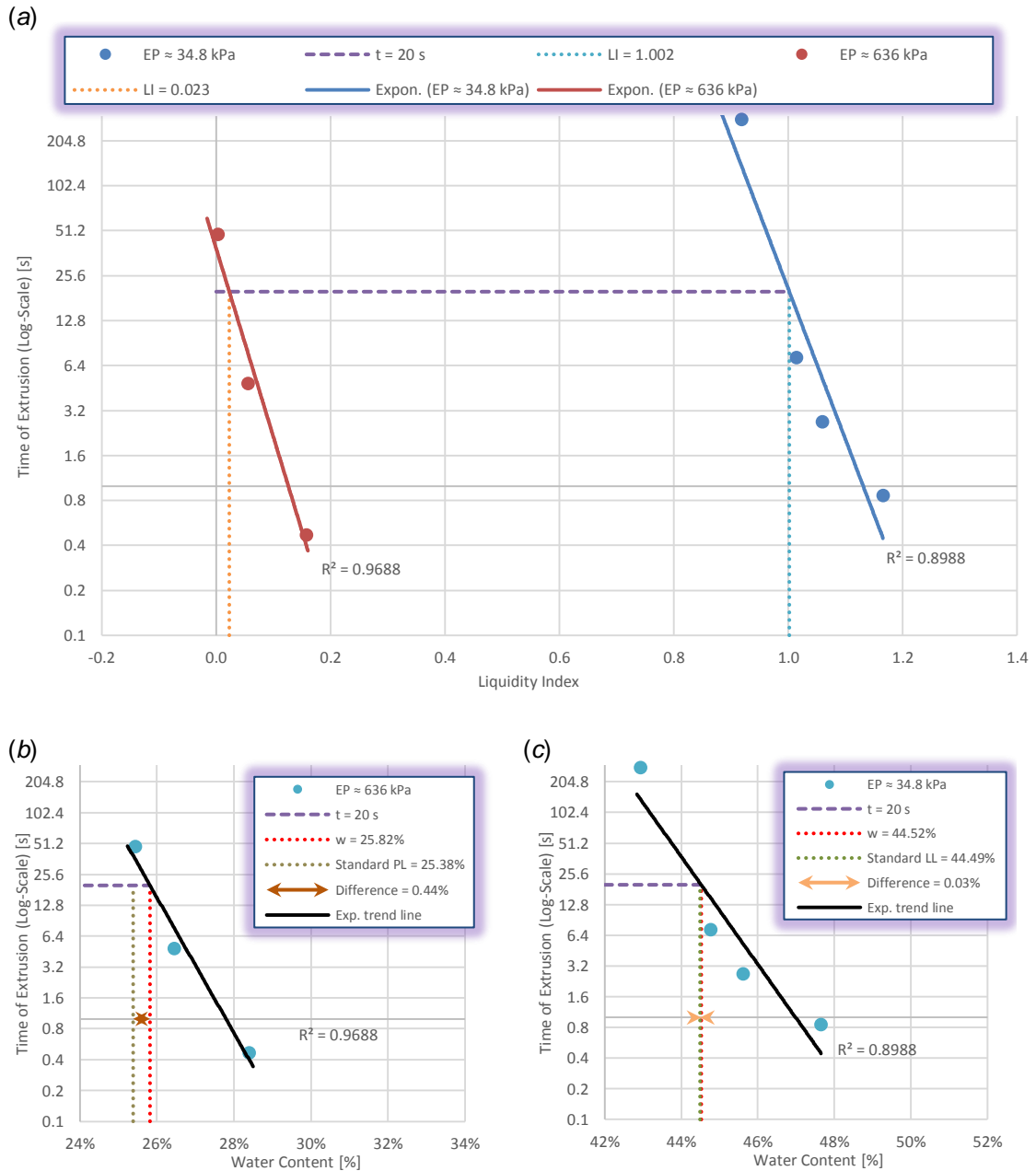


Figure 4-9: Soil 5 behaviour determination graphs by the second prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-9: Soil behaviour determination tests on soil 6 by the second prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
90.06	34.76	42.86%	1.19	0.53	91.76	34.76	38.19%	0.94	31.44
91.23	34.76	39.91%	1.03	2.65	99.15	635.98	23.26%	0.13	1.12
90.96	34.76	39.77%	1.02	2.06	99.82	635.98	22.10%	0.07	2.78
91.11	34.76	39.38%	1.00	2.37	99.78	635.98	21.91%	0.06	9.50
91.97	34.76	38.72%	0.97	8.72	101.65	635.98	21.57%	0.04	33.59
91.49	34.76	38.42%	0.95	11.50					

* Decimal values of water content have been considered for more accuracy.

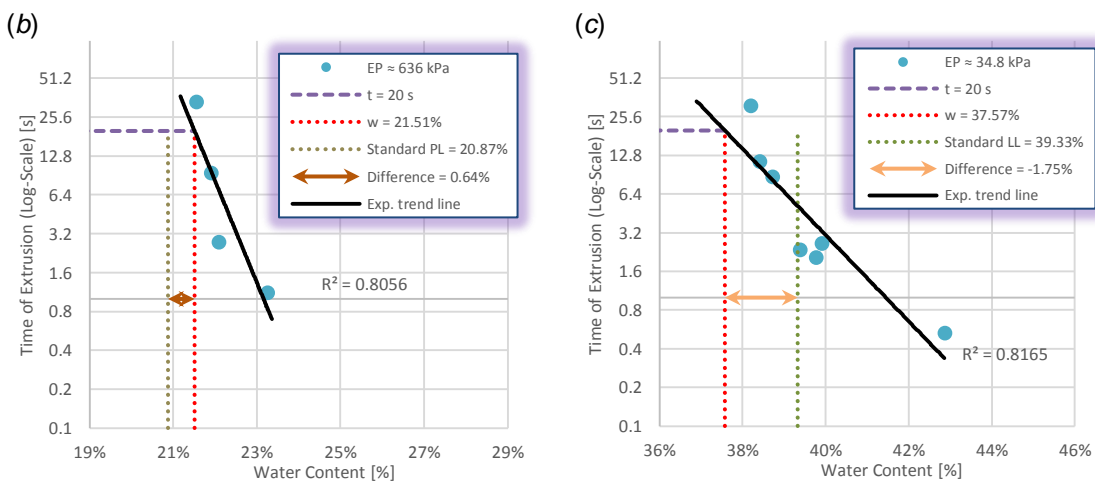
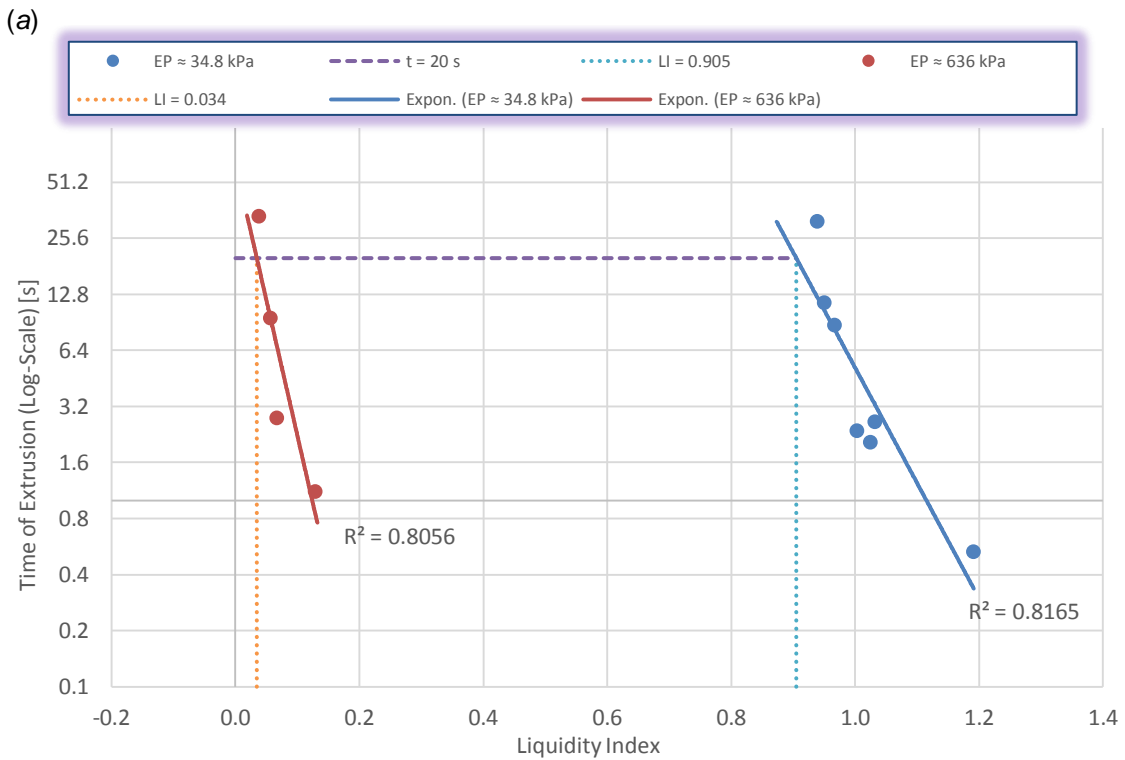


Figure 4-10: Soil 6 behaviour determination graphs by the second prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

4.4 Manafi's third prototype

It was tried to enhance the performance of the previous prototype by covering the minor mechanical problems and increasing operator's convenience. As explained in chapter 3, the chosen ER^{-1} for determination of LL and PL are 14.3% and 57.1% respectively. Likewise calibration of the previous prototype, the third prototype was calibrated with extrusion pressures of 35.64 kPa and 495.42 kPa, and with extrusion time of 10 s and 30 s for determination of liquid and plastic limits respectively. Considering the change in the extrusion ratio and the design of the prototype, the extrusion pressures and the extrusion time is different with the previous prototype.

4.4.1 Soil behaviour determination by the third prototype

The soil behaviour determination tests by proposed method have been done on eight soil samples. The results are presented in Table 4-10 to Table 4-17 and Figure 4-11 to Figure 4-18.

Table 4-10: Soil behaviour determination tests on soil 1 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
92.76	35.64	33.07%	0.94	78.22	103.77	495.42	20.03%	0.02	25.03
92.27	35.64	33.84%	1.00	23.63	102.64	495.42	20.53%	0.05	10.31
91.5	35.64	34.44%	1.04	5.78	103.51	495.42	21.75%	0.14	0.53
91.18	35.64	35.48%	1.11	1.66					

* Decimal values of water content have been considered for more accuracy.

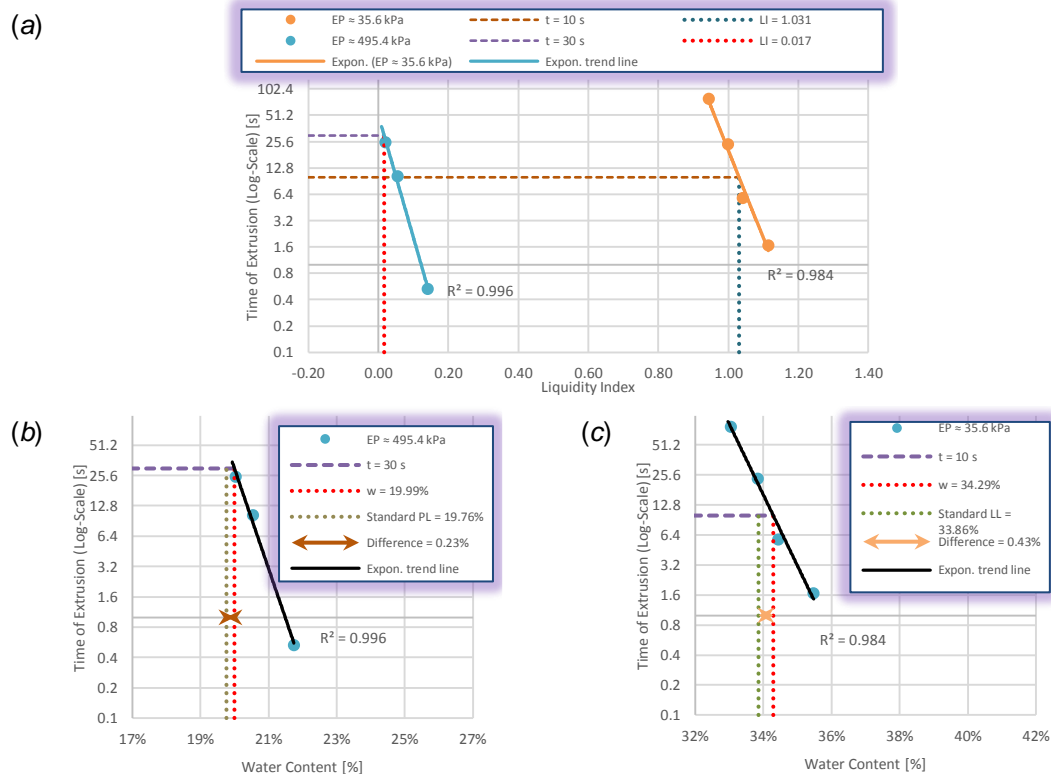


Figure 4-11: Soil 1 behaviour determination graphs by the third prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-11: Soil behaviour determination tests on soil 2 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
82.21	35.64	53.75%	0.90	67.63	96.42	495.42	30.07%	-0.03	40.82
82.18	35.64	54.53%	0.94	16.35	94.48	495.42	30.29%	-0.02	7.41
81.93	35.64	55.90%	0.99	9.19	93.92	495.42	31.82%	0.04	3.57
					91.86	495.42	34.50%	0.15	0.47

* Decimal values of water content have been considered for more accuracy.

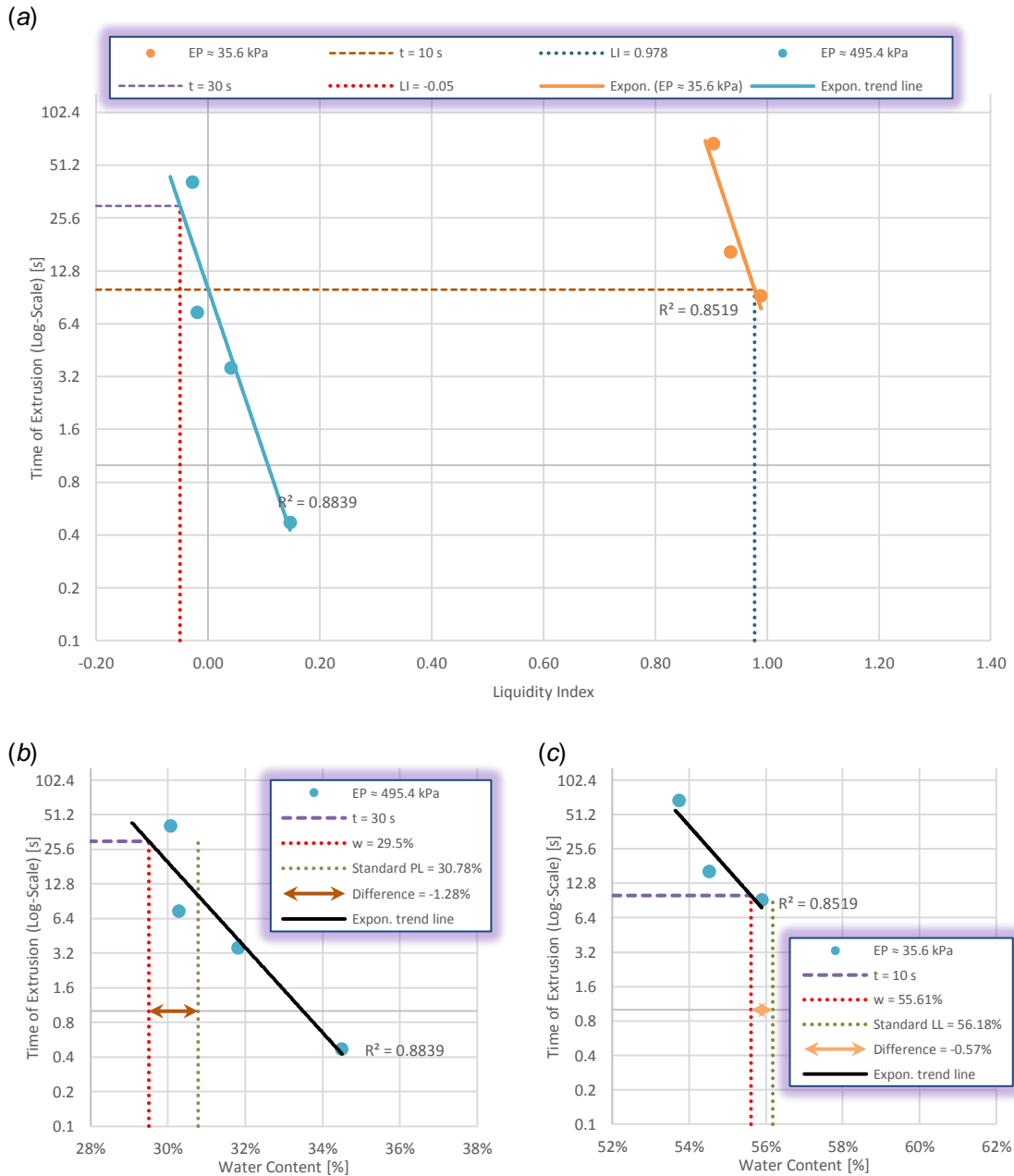


Figure 4-12: Soil 2 behaviour determination graphs by the third prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-12: Soil behaviour determination tests on soil 3 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
80.95	35.64	57.82%	0.89	38.32	92.32	495.42	28.33%	0.05	23.19
79.99	35.64	58.93%	0.92	8.34	94.92	495.42	29.56%	0.09	8.44
79.28	35.64	62.33%	1.02	1.62	95.9	495.42	30.27%	0.11	3.22
79.03	35.64	64.18%	1.07	1.31	94.79	495.42	30.80%	0.12	0.93
96.94	495.42	27.32%	0.02	97.85	93.53	495.42	31.98%	0.16	0.43

* Decimal values of water content have been considered for more accuracy.

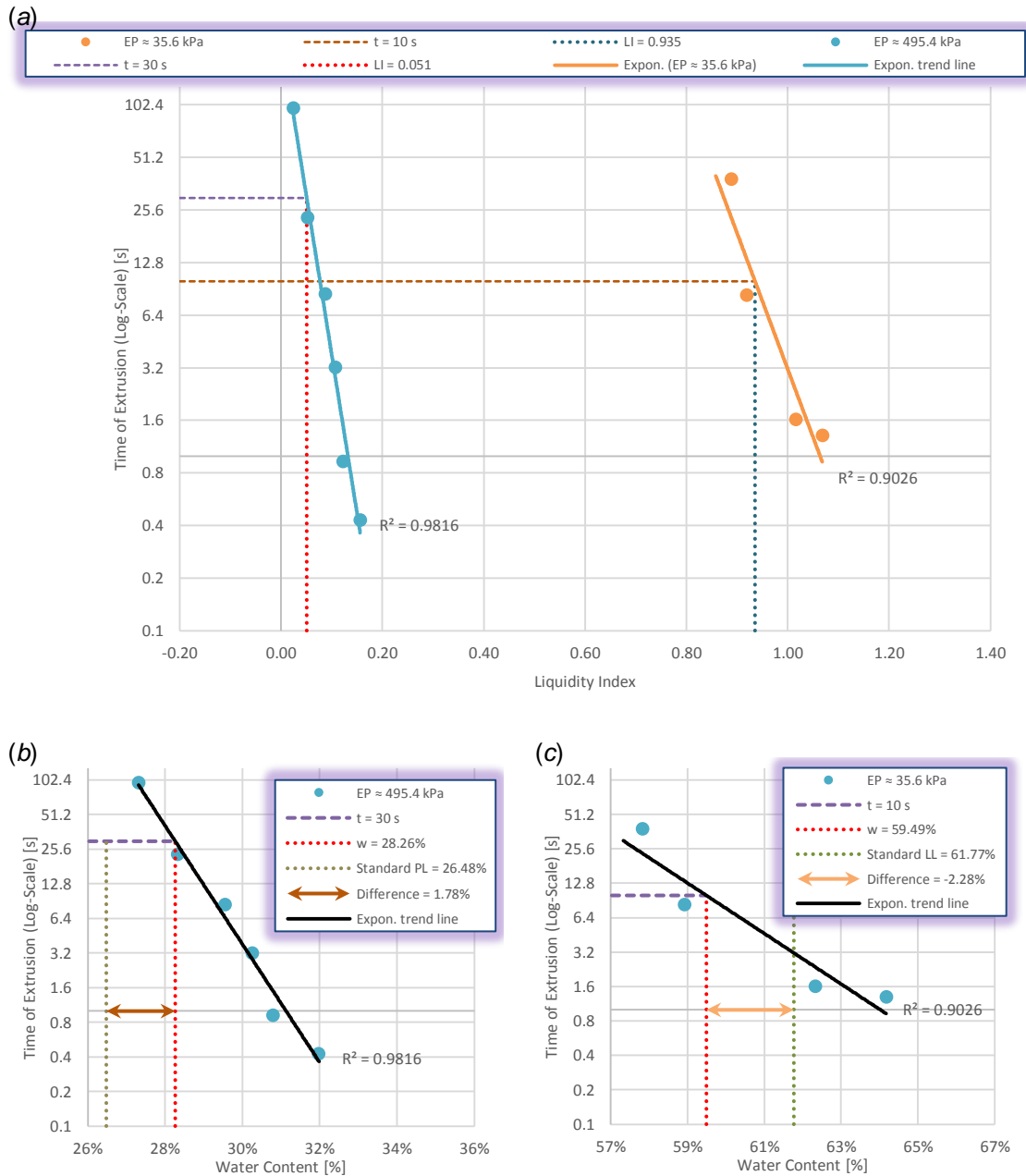


Figure 4-13: Soil 3 behaviour determination graphs by the third prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-13: Soil behaviour determination tests on soil 4 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
94.31	35.64	31.14%	1.09	28.72	106.63	495.42	19.78%	-0.03	50.72
94.07	35.64	32.30%	1.20	5.53	105.84	495.42	20.26%	0.01	5.62
93.47	35.64	32.74%	1.25	2.32	106.87	495.42	20.27%	0.02	3.12
93.13	35.64	33.39%	1.31	1.25	106.05	495.42	20.62%	0.05	0.41

* Decimal values of water content have been considered for more accuracy.

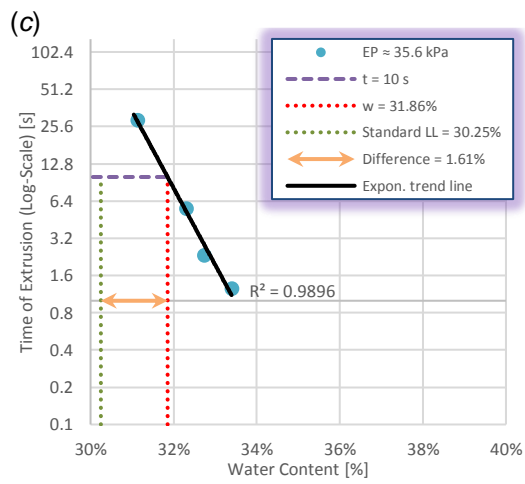
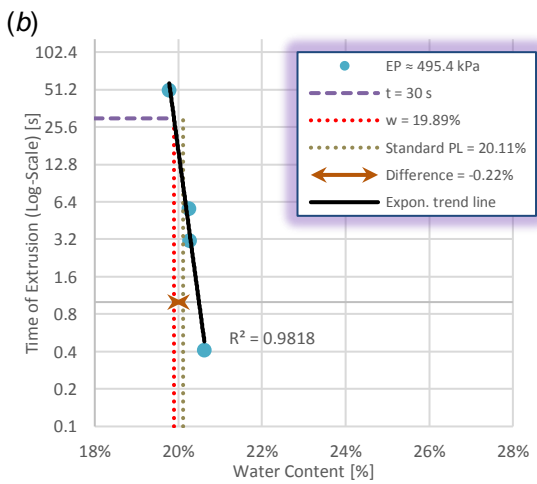
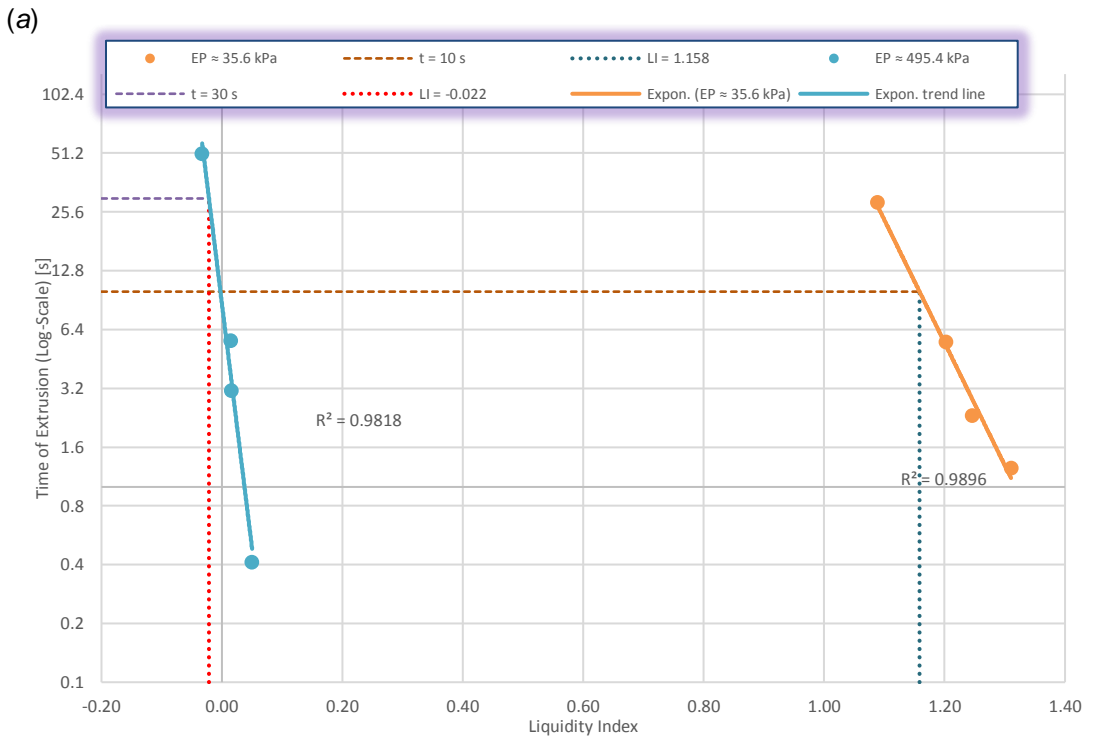


Figure 4-14: Soil 4 behaviour determination graphs by the third prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-14: Soil behaviour determination tests on soil 7 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
86.01	35.64	44.60%	0.97	42.50	99.98	495.42	24.82%	-0.06	75.59
85.9	35.64	45.50%	1.01	14.62	100.1	495.42	25.71%	-0.02	5.84
85.45	35.64	46.11%	1.05	5.35	98.95	495.42	26.73%	0.04	0.81
					96.53	495.42	28.97%	0.15	0.38

* Decimal values of water content have been considered for more accuracy.

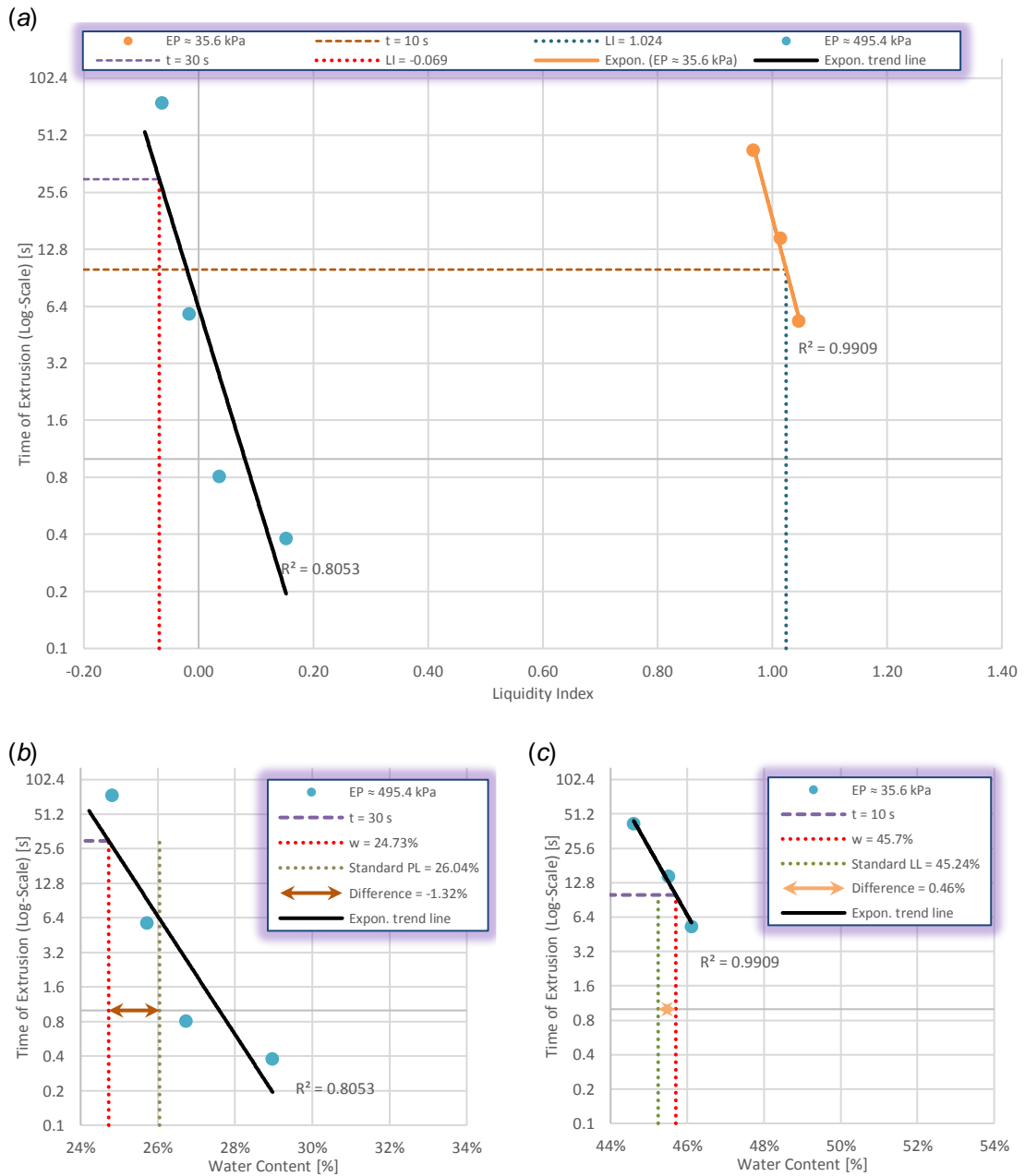


Figure 4-15: Soil 7 behaviour determination graphs by the third prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-15: Soil behaviour determination tests on soil 8 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
88.12	35.64	40.60%	0.87	29.63	101.02	495.42	21.98%	0.08	97.25
88.09	35.64	41.67%	0.91	6.79	99.39	495.42	22.54%	0.10	22.45
87.73	35.64	42.75%	0.96	1.71	102.68	495.42	22.86%	0.11	2.43
86.13	35.64	44.75%	1.04	0.75	100.5	495.42	22.89%	0.11	10.13

* Decimal values of water content have been considered for more accuracy.

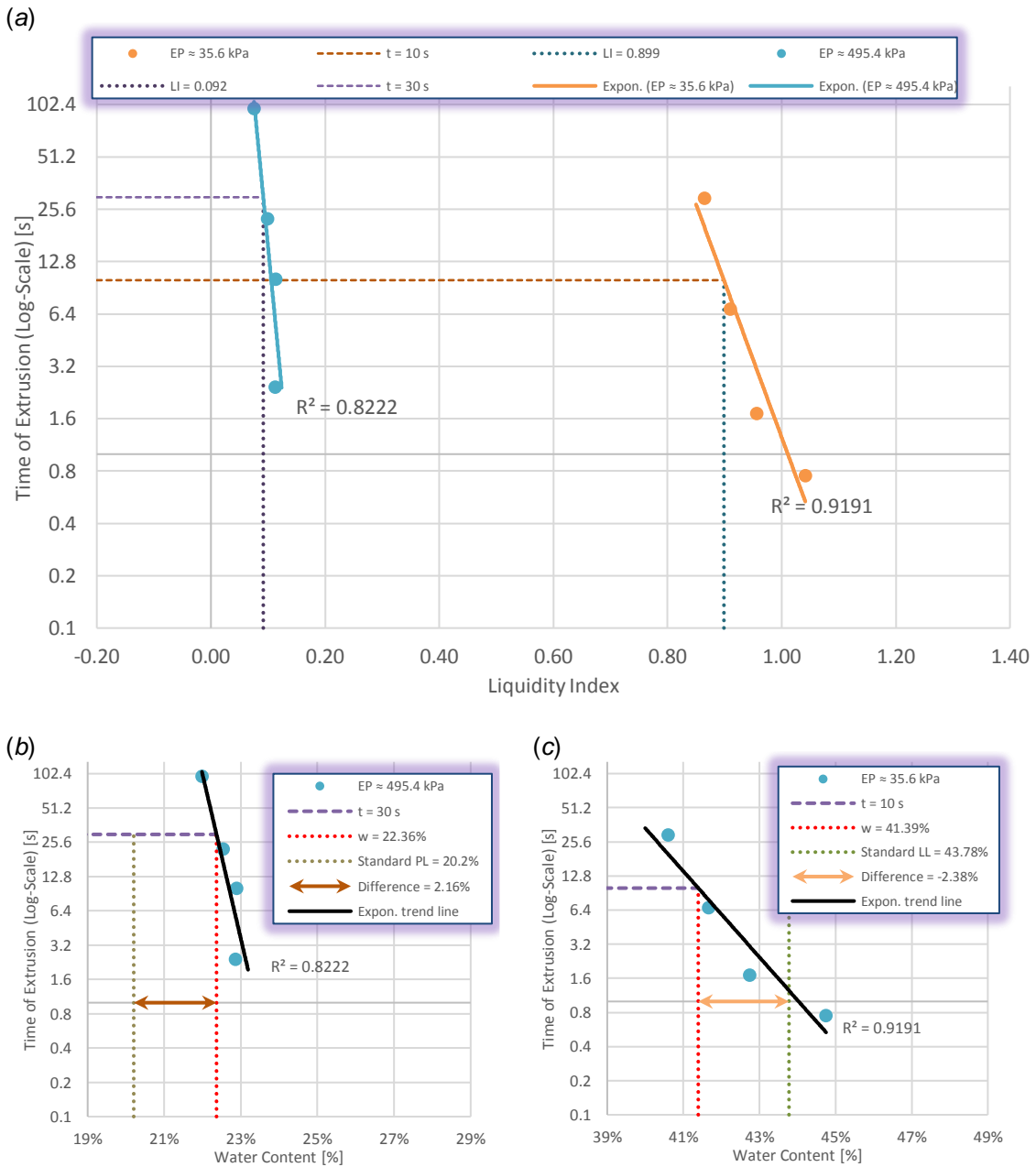


Figure 4-16: Soil 8 behaviour determination graphs by the third prototype: a) *t* vs. *LI* for various extrusion pressures; b) *t* vs. *w*% for plastic limit; c) *t* vs. *w*% for liquid limit.

Table 4-16: Soil behaviour determination tests on soil 9 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
89.9	35.64	38.10%	1.00	47.34	102.64	495.42	22.35%	0.05	62.97
89.95	35.64	38.59%	1.03	23.38	102.97	495.42	22.81%	0.08	23.03
89.69	35.64	38.99%	1.05	16.35	103.13	495.42	23.02%	0.09	3.65
88.72	35.64	40.33%	1.14	1.87	101.98	495.42	23.94%	0.14	0.66

* Decimal values of water content have been considered for more accuracy.

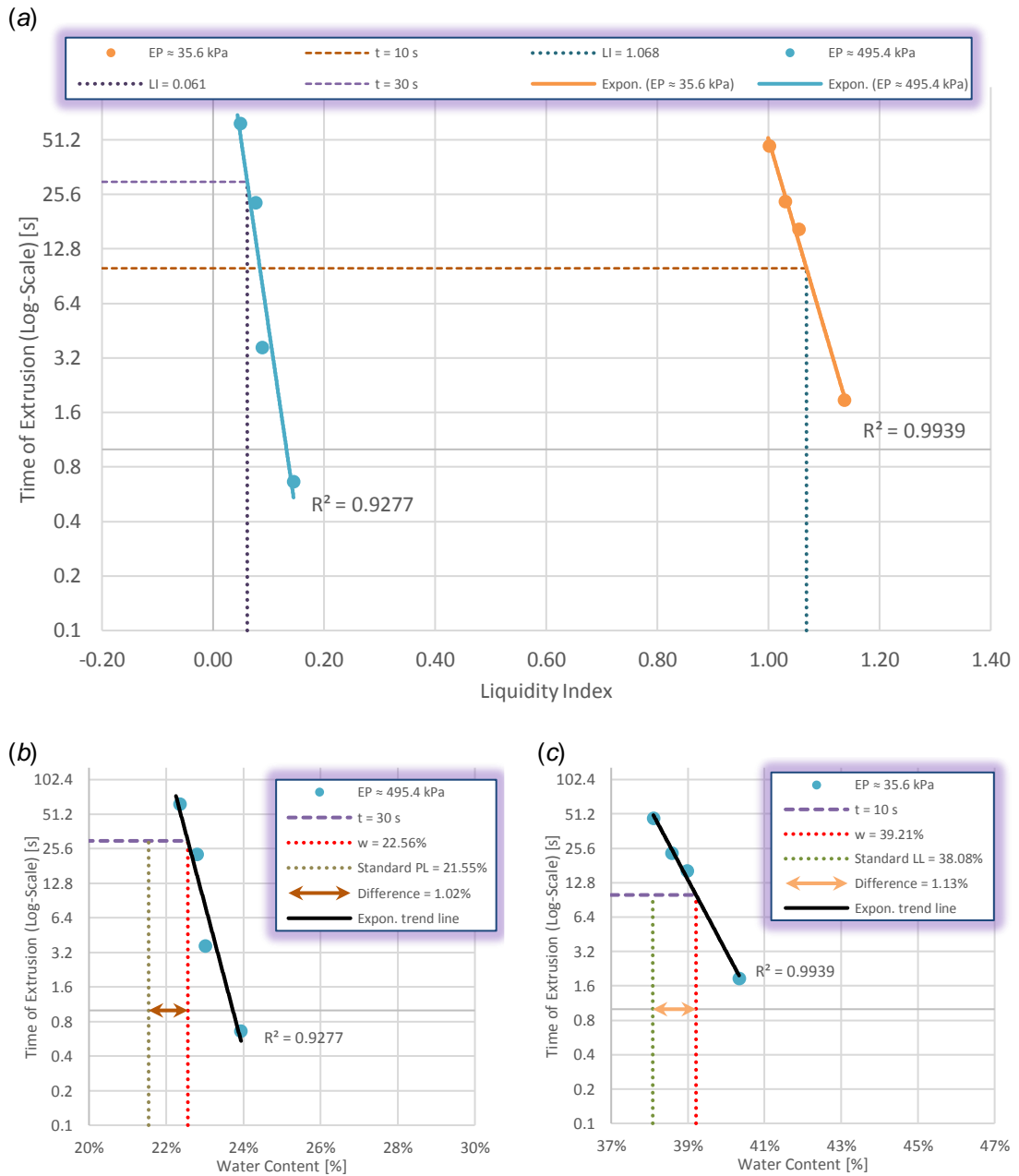


Figure 4-17: Soil 9 behaviour determination graphs by the third prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

Table 4-17: Soil behaviour determination tests on soil 10 by the third prototype

Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]	Mass of specimen [g]	Extrusion pressure [kPa]	Water content* [%]	Liquidity Index	Time of extrusion [s]
90.83	35.64	33.39%	0.85	29.75	103.15	495.42	20.67%	0.12	30.15
90.84	35.64	33.98%	0.89	12.98	101.58	495.42	21.04%	0.14	13.84
90.6	35.64	35.04%	0.95	4.38	102.23	495.42	21.79%	0.18	1.44
90.3	35.64	35.82%	0.99	2.01					

* Decimal values of water content have been considered for more accuracy.

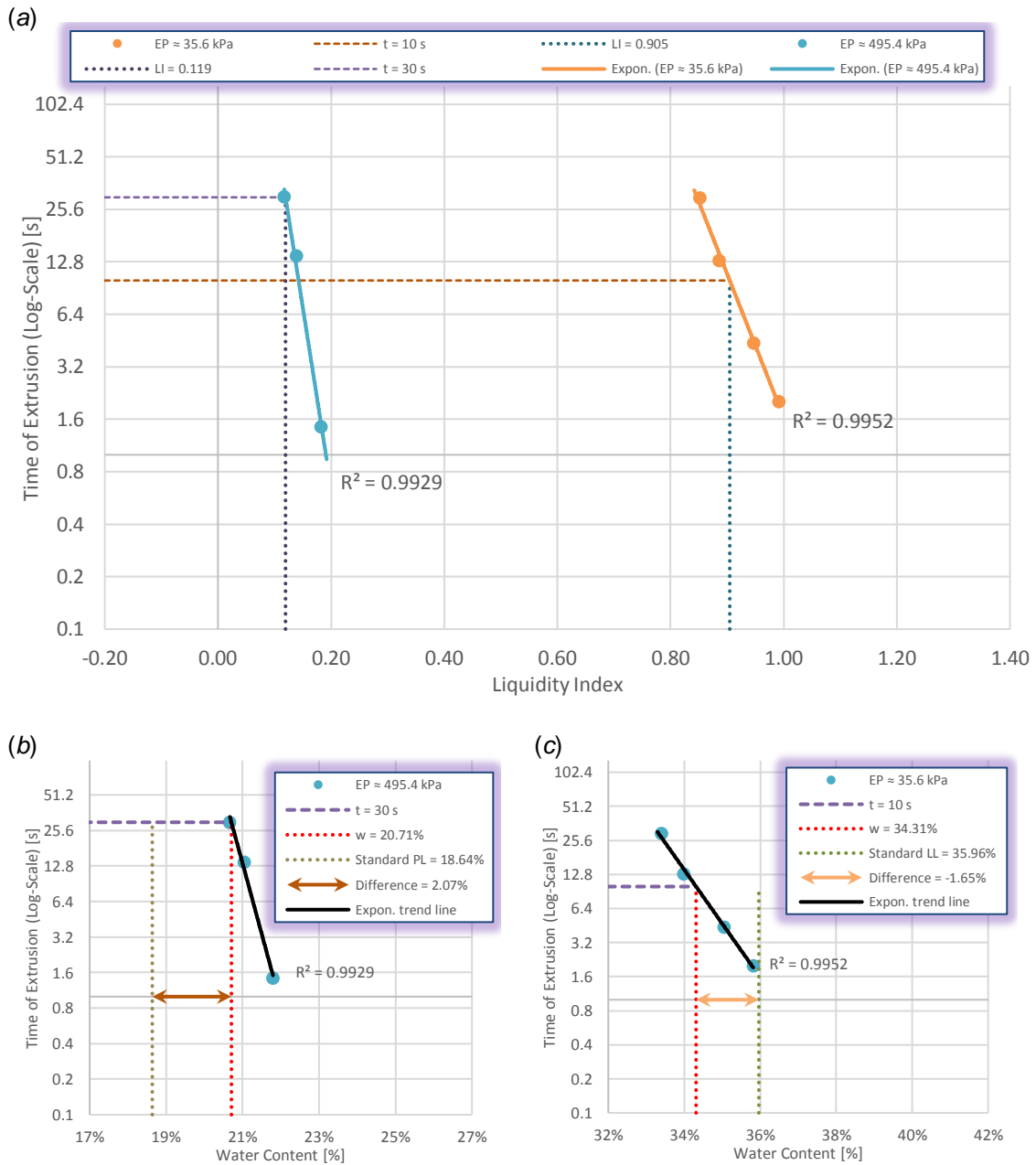


Figure 4-18: Soil 10 behaviour determination graphs by the third prototype: a) t vs. LI for various extrusion pressures; b) t vs. $w\%$ for plastic limit; c) t vs. $w\%$ for liquid limit.

4.5 Data analysis

Ten different soils from various soil types (CL, MH, and CH) have been studied. The soils studied under 6 different extrusion pressures (about 34.8, 35.6, 144, 253.3, 495.4, and 635.9 kPa) with various extrusion ratios (ER^{-1} about 14.3%, 30.6%, and 57.1%).

Although the general soil behaviour curves have some data noise in soil behaviour graphs, the general paths are obviously distinctive with high values of the Pearson product moment correlation coefficient (R^2) through data points as shown in the graphs. The data noise could be due to several reasons:

- Existing mechanical problems of current prototype due to fabrication limitations;
- Simplification of test method;
- Inaccuracy in manual determination of extrusion time;
- Inaccuracy of standard test methods for determination of soil plasticity and water content.

All of these sources of data noise will be discussed in details later. The prototype performed very well and showed almost the same general behaviour for the investigated soils.

Table 4-18 presents the results obtained by current standard methods and the proposed method by the second prototype. Based on the gathered data, it can be said that the second prototype (with $ER^{-1} = 30.6\%$ and $t = 20$ s) is calibrated for determination of soils consistency limits with LI equal to 0.995 and 0.025 with extrusion pressures of 34.76 and 635.98 kPa respectively. A sample calculation for soil 1 is provided to have a better understanding of the contents of the table:

Standard PL value: 19.76%; Standard LL value: 33.86% (based on the discussions in chapters 2 and 3, these values are not necessarily the real plastic limits of soil, but are close to it).

Then, several specimens with water contents around limit states were prepared. These specimens were tested with the second prototype to observe the behaviour of soil around the limit states (the results are provided in Table 4-4 and Figure 4-5). It was determined that the soil with water contents of 21.05% ($LI = 0.092$ based on standard methods) and 33.75% ($LI = 1.032$ based on standard methods) would be extruded in 20 s by the second prototype ($w_{PLD}\%$ and $w_{LLD}\%$ in Table 4-18). Hence it would be possible to calculate the workability of soil at these water contents as explained in chapter 3. Simplified physical model of the problem is shown in Figure 4-19. Workability of soil with 33.75% water content around real LL can be calculated as follows:

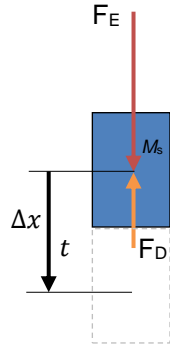


Figure 4-19: Simplified physical model of the problem

$$F_E = M_E \times g = 3.1853 \times 9.81 = 31.248 \text{ N} \quad (4-1)$$

where F_E is the extrusion force, M_E is the extrusion load mass that causes the extrusion pressure, and g is the acceleration due to gravity.

$$a = \frac{2\Delta x}{t^2} = \frac{2 \times 50}{20^2} = 0.25 \text{ mm/s}^2 \quad (4-2)$$

where a is the average acceleration of soil specimen, Δx is the magnitude of the soil specimen displacement, and t is the time of extrusion.

$$F = F_E - F_D = M_s \times a = 91 \times 0.25 = 0.023 \text{ N} \quad (4-3)$$

where F is the resultant force causing soil specimen acceleration, F_E is the magnitude of extrusion force, F_D is the magnitude of force for soil deformation, and M_s is the mass of soil specimen.

$$F_D = F_E - F = 31.248 - 0.023 = 31.225 \text{ N} \quad (4-4)$$

The workability of soil paste can be measured by following equations:

$$P_D = \frac{W_D}{t} = \frac{F_D \Delta x}{t} = \frac{1561.25}{20} = 78.06 \text{ mJ/s} \quad (4-5)$$

where W_D is the work done for deformation of soil. The workability of soil at any other water content can be calculated with the same procedure. The results are presented in Table 4-20.

It was observed that all soils with almost the same workability are very close to the standard limit states. However, standard methods have their own deficiencies and are not completely decisive as well. Hence, the average of all soils' LIs was considered as the benchmark to correlate to the workability of soil. Hence, all soils with the same workability ($P_D = 78.06 \text{ mJ/s}$) would be at $LI = 0.995$, which is very close to LL and can be considered as LL . Correspondingly, all soils with the same workability ($P_D = 1429.09 \text{ mJ/s}$) would be at $LI = 0.025$, which is very close to PL and can be considered as PL .

Although it is possible to find the exact workability pertinent to exact $LI = 0$ and 1 by changing the extrusion force and time of extrusion by trial and error, it is also possible to calculate real limit state by solving simultaneous equations of the corresponding LIs :

$$\begin{cases} 0.025 = \frac{21.05\% - PL}{LL - PL} \\ 0.995 = \frac{33.31\% - PL}{LL - PL} \end{cases} \xrightarrow{\text{yields}} PL = 20.71\%; LL = 34.38\% \quad (4-6)$$

These values are presented as Manafi LL and PL in Table 4-19. The difference between limit states by proposed method and standard methods are presented in the table as well as the difference between water contents at $LI = 0.025$ and 0.995 (simple method) with standard limit states. Finally, the difference between real limit states (Manafi method) and the simple method shows that the difference is ignorable and it is possible to determine the real limit states of soil by the simple method.

Table 4-18: Soil behaviour determination tests by the second prototype

Soil No.	Thread rolling PL	Fall cone LL	$w_{PLD}\%$ ¹	LI for PL determination ²	Average LI for PL ³	$w_{LLD}\%$ ⁴	LI for LL determination ⁵	Average LI for LL ⁶
1	19.76%	33.86%	21.05%	0.092	0.025	34.31%	1.032	0.995
2	30.78%	56.18%	29.69%	-0.043		55.26%	0.964	
3	26.48%	61.77%	27.45%	0.027		58.32%	0.902	
4	20.11%	30.25%	20.27%	0.015		31.90%	1.163	
5	25.38%	44.49%	25.82%	0.023		44.52%	1.002	
6	20.87%	39.33%	21.51%	0.034		37.57%	0.905	

1- Soils had the same workability for plastic limit determination at this water content.

2- Liquidity index of soil at $w_{PLD}\%$ based on standard methods.

3- Average of determined LIs (calibrated performance of prototype) for plastic limit determination.

4- Soils had the same workability for liquid limit determination at this water content.

5- Liquidity index of soil at $w_{LLD}\%$ based on standard methods.

6- Average of determined LIs (calibrated performance of prototype) for liquid limit determination.

Table 4-19: Analyzing accuracy of the second prototype for determination of limit states

Soil No.	Manafi PL ¹	Manafi LL ¹	(Manafi PL - STD PL) ²	(Manafi LL - STD LL) ²	(w_{PLD} - STD PL) ³	(w_{LLD} - STD LL) ³	(Manafi PL - w_{PLD}) ⁴	(Manafi LL - w_{LLD}) ⁴
1	20.71%	34.38%	0.95%	0.52%	1.29%	0.45%	-0.34%	0.07%
2	29.03%	55.41%	-1.75%	-0.77%	-1.09%	-0.92%	-0.66%	0.14%
3	26.66%	58.48%	0.18%	-3.29%	0.97%	-3.45%	-0.79%	0.17%
4	19.97%	31.97%	-0.14%	1.72%	0.16%	1.65%	-0.30%	0.06%
5	25.34%	44.63%	-0.04%	0.13%	0.44%	0.03%	-0.48%	0.10%
6	21.10%	37.66%	0.22%	-1.66%	0.64%	-1.75%	-0.41%	0.09%

1- Soils plasticity determination based on proposed method (real limit states) according to Eq. (4-6).

2- Difference of proposed method in comparison to standard methods.

3- Difference of simple method in comparison to standard methods.

4- Difference of proposed method in comparison to the simple method.

Table 4-20: Workability measurements of soils by the second prototype

Soil No.	Liquid limit determination				Plastic limit determination			
	M_s [g] ¹	F_D [N] ²	W_D [mJ] ³	P_D [mJ/s] ⁴	M_s [g] ¹	F_D [N] ²	W_D [mJ] ³	P_D [mJ/s] ⁴
1	91	31.23	1561.25	78.06	101	571.63	28581.74	1429.09
2	82	31.23	1561.36	78.07	94	571.64	28581.82	1429.09
3	87	31.23	1561.30	78.07	96	571.64	28581.80	1429.09
4	97	31.22	1561.18	78.06	104	571.63	28581.70	1429.08
5	89	31.23	1561.28	78.06	97	571.64	28581.79	1429.09
6	91	31.23	1561.25	78.06	100	571.63	28581.75	1429.09

1- Soil specimen mass in the container.

2- Soil deformation force according to Eq. (4-4)

3- Soil deformation work according to Eq. (4-5)

4- Soil deformation power (soil workability) according to Eq. (4-5)

Figure 4-20 demonstrates a comparison between results obtained by proposed method and standard methods.

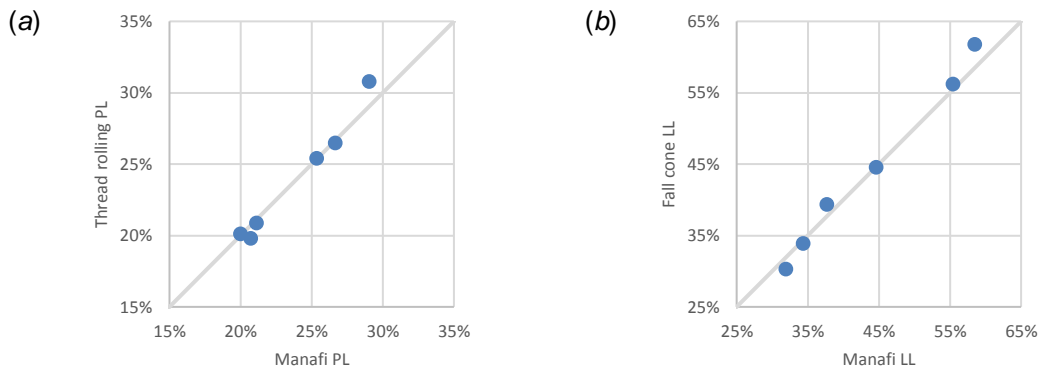


Figure 4-20: Comparison between results obtained by proposed method with the second prototype and conventional methods: a) Plastic limit; b) Liquid limit.

Graph of extrusion pressure against average liquidity index of soils with the second prototype is shown in Figure 4-21. This graph shows that it is required to increase the extrusion pressure of prototype exponentially for extrusion of soils with lower water contents. This exponential increase rate explains the different slope of graphs from wet side to dry side of investigated soils. This graph also shows the capability of proposed method for direct determination of any arbitrary consistency state by applying proper extrusion pressure and ER^{-1} .

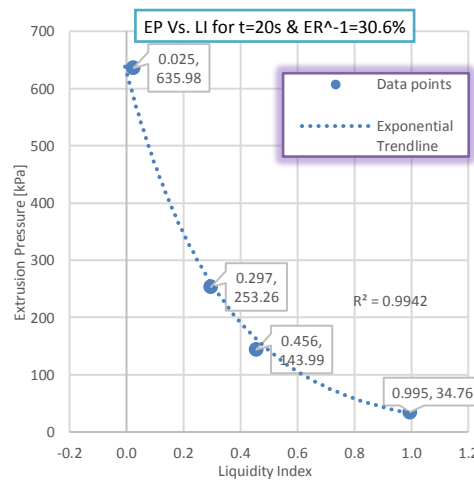


Figure 4-21: Extrusion pressure against average liquidity index of investigated soils with the second prototype

The same procedure followed by the third prototype. Table 4-21 presents the results obtained by current standard methods and the proposed method by the third prototype. Based on the gathered data, it can be said that the third prototype (with $ER^{-1} = 14.3\%$ and $t = 30$ s; and 57.1% , and $t = 10$ s) is calibrated for determination of soils consistency limits with LI equal to 1.000 and 0.025 with extrusion pressures of 35.64 and 495.42 kPa respectively. The comparison between the results of proposed method and standard

methods are provided in Table 4-22. The workability of soil at benchmark LIs are also presented in Table 4-23.

Table 4-21: Soil behaviour determination tests by the third prototype

Soil No.	Thread rolling PL	Fall cone LL	$w_{PLD}\%$ ¹	LI for PL determination ²	Average LI for PL ³	$w_{LLD}\%$ ⁴	LI for LL determination ⁵	Average LI for LL ⁶
1	19.76%	33.86%	19.99%	0.017	0.025	34.29%	1.031	1.000
2	30.78%	56.18%	29.50%	-0.050		55.61%	0.978	
3	26.48%	61.77%	28.26%	0.051		59.49%	0.935	
4	20.11%	30.25%	19.89%	-0.022		31.86%	1.158	
7	26.04%	45.24%	24.73%	-0.069		45.70%	1.024	
8	20.20%	43.78%	22.36%	0.092		41.39%	0.899	
9	21.55%	38.08%	22.56%	0.061		39.21%	1.068	
10	18.64%	35.96%	20.71%	0.119		34.31%	0.905	

1- Soils had the same workability for plastic limit determination at this water content.

2- Liquidity index of soil at $w_{PLD}\%$ based on standard methods.

3- Average of determined LIs (calibrated performance of prototype) for plastic limit determination..

4- Soils had the same workability for liquid limit determination at this water content.

5- Liquidity index of soil at $w_{LLD}\%$ based on standard methods.

6- Average of determined LIs (calibrated performance of prototype) for liquid limit determination.

Table 4-22: Analysing accuracy of the third prototype for determination of limit states

Soil No.	Manafi PL ¹	Manafi LL ¹	(Manafi PL - STD PL) ²	(Manafi LL - STD LL) ²	(w_{PLD} - STD PL) ³	(w_{LLD} - STD LL) ³	(Manafi PL - w_{PLD}) ⁴	(Manafi LL - w_{LLD}) ⁴
1	19.63%	34.30%	-0.13%	0.44%	0.23%	0.43%	-0.37%	0.00%
2	28.84%	55.62%	-1.94%	-0.56%	-1.28%	-0.57%	-0.67%	0.01%
3	27.46%	59.50%	0.98%	-2.27%	1.78%	-2.28%	-0.80%	0.01%
4	19.59%	31.86%	-0.52%	1.61%	-0.22%	1.61%	-0.31%	0.00%
7	24.19%	45.71%	-1.85%	0.47%	-1.32%	0.46%	-0.54%	0.01%
8	21.88%	41.40%	1.67%	-2.38%	2.16%	-2.38%	-0.49%	0.00%
9	22.14%	39.22%	0.59%	1.13%	1.02%	1.13%	-0.43%	0.00%
10	20.36%	34.31%	1.72%	-1.65%	2.07%	-1.65%	-0.35%	0.00%

1- Soils plasticity determination based on proposed method (real limit states) according to Eq. (4-6).

2- Difference of proposed method in comparison to standard methods.

3- Difference of simple method in comparison to standard methods.

4- Difference of proposed method in comparison to the simple method.

Table 4-23: Workability measurements of soils by the third prototype

Soil No.	Liquid limit determination				Plastic limit determination			
	M_s [g] ¹	F_D [N] ²	W_D [mJ] ³	P_D [mJ/s] ⁴	M_s [g] ¹	F_D [N] ²	W_D [mJ] ³	P_D [mJ/s] ⁴
1	92	31.79	1589.50	158.95	104	443.20	22160.14	738.67
2	82	31.80	1590.00	159.00	95	443.20	22160.19	738.67
3	81	31.80	1590.05	159.01	93	443.20	22160.20	738.67
4	94	31.79	1589.40	158.94	106	443.20	22160.13	738.67
7	85	31.80	1589.85	158.99	100	443.20	22160.16	738.67
8	88	31.79	1589.70	158.97	100	443.20	22160.16	738.67
9	89	31.79	1589.65	158.97	103	443.20	22160.14	738.67
10	91	31.79	1589.55	158.96	103	443.20	22160.14	738.67

1- Soil specimen mass in the container.

2- Soil deformation force according to Eq. (4-4)

3- Soil deformation work according to Eq. (4-5)

4- Soil deformation power (soil workability) according to Eq. (4-5)

According to Table 4-22, the most difference between the results obtained by proposed method and standard method is in the case of soil 8 for liquid limit with the difference of 2.38% which is negligible in comparison to the accuracy of current standard methods, as will be discussed later in 4.5.4.3. According to Figure 4-3, most of this soil comprises of clay (50.6%) and silt (45.3%). These soils are usually among problematic soils and accurate measurement of their plasticity is difficult based on standard methods. Hence, it is not strange to observe the difference between the results obtained by current

standard methods and the proposed method. Soil 2 also shows the most difference among the other soils for determination of plastic limit in proposed method (1.94%). According to Figure 4-3, this soil has the most sand portion among the soils (28.7%). This difference might be due to difficulty in determination of plastic limit of sandy soils by thread rolling method. It should be noted that these differences are due to inaccuracy of current standard methods together with water content determination by proposed method. Figure 4-22 demonstrates a comparison between results obtained by proposed method and standard methods. The trend lines in the graphs show a good compatibility between the results obtained by the standard methods and the proposed method for the plasticity range of studied soils.

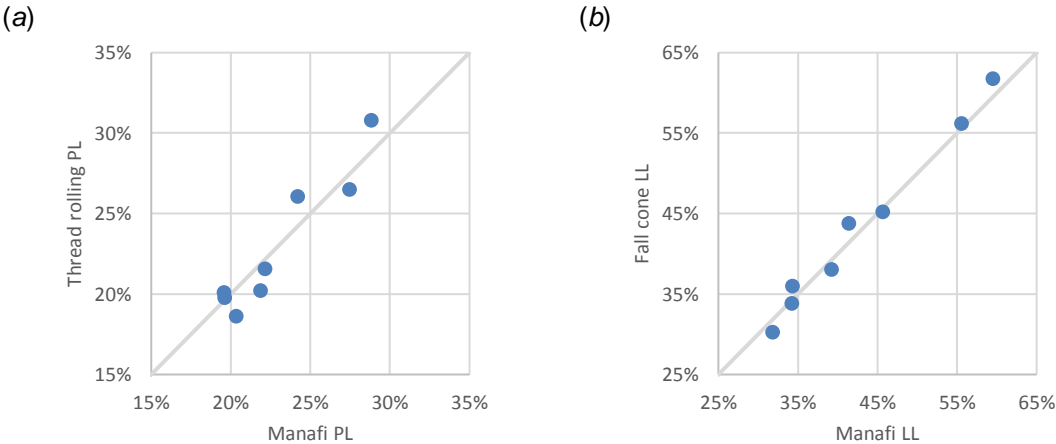


Figure 4-22: Comparison between results obtained by proposed method with the third prototype and conventional methods: a) Plastic limit; b) Liquid limit.

4.5.1 Study accuracy of proposed method

It is possible to evaluate the accuracy of proposed method in comparison to standard methods. R^2 value (coefficient of multiple determination) is a statistical measure of nearness of data points to the fitted regression line (Frost 2013). In general, the higher R^2 value, the better the regression fits the data points. Figure 4-23 compares the results obtained by proposed method and standard methods within the range of investigated soils limit states. Considering the R^2 value as a measure of the accuracy of the method for providing more consistent results, the higher R^2 value, the more consistent results obtained by the method. According to values Figure 4-23, R^2 values of proposed method in both of limit state determinations are more than standard methods. Hence, the proposed method provides more consistent results compare to standard methods.

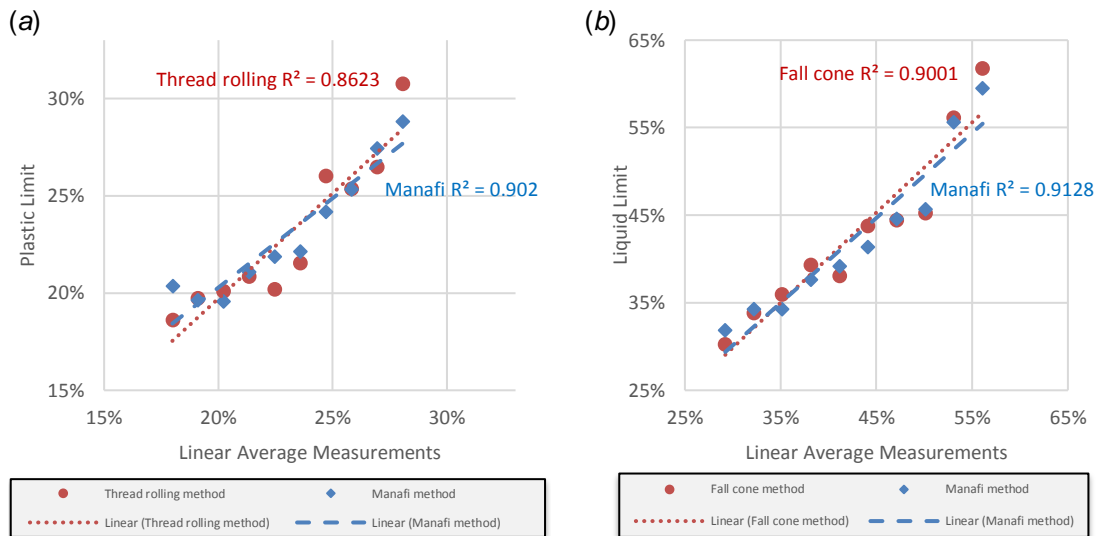


Figure 4-23: Comparison between results obtained by proposed method and standard methods: a) Plastic limit; b) Liquid limit.

4.5.2 Study effect of improvement in prototype

Improvements applied on the third prototype not only enhanced the mechanical performance of the prototype but also made the experiment procedure easier for the operator. Soils number 1 to 4 were tested by both of the prototypes. Table 4-24 shows the R^2 values for the tested soils on both of the prototypes. Rise in R^2 values in the third prototype shows an average about 9% improvement in the performance of the third prototype.

Table 4-24: R^2 values for fitted curves in different prototypes

Soil No.	Second Prototype		Third Prototype	
	LL Det. R^2	PL Det. R^2	LL Det. R^2	PL Det. R^2
1	0.696	0.9696	0.984	0.996
2	0.857	0.7402	0.8519	0.8839
3	0.8815	0.8138	0.9026	0.9816
4	0.9709	0.9725	0.9896	0.9818

4.5.3 Study effect of change in extrusion ratio

The second prototype had a fixed extrusion ratio ($ER^{-1} = 30.6\%$), but the third prototype had different extrusion ratios for determination of liquid and plastic limits ($ER^{-1} = 14.3\%$ and 57.1% respectively). Table 4-25 compares the results obtained by both of the prototypes. The results show that the change in extrusion pressure is not proportionate to the change in ER^{-1} , but the change in soil deformation power (workability of soil) is inversely proportionate to the change in ER^{-1} with almost the same ratio. It should be noted that the workability of soil at a specific state is constant but it will change by changing any design of the apparatus. In other words, the soil deforms easier or harder by changing in apparatus design.

Table 4-25: Effect of change in extrusion ratio in the third prototype

	<i>LL</i>	<i>PL</i>
(ER^{-1}_3 / ER^{-1}_2)	0.47	1.87
Change in ER^{-1}	-53.3%	86.6%
Change in extrusion pressure (EP)	2.5%	-22.1%
Change in soil deformation power (P_D)	103.6%	-48.3%
$(\text{Change in } ER^{-1} / \text{Change in } P_D)$	-0.51	-1.79

4.5.4 Sources of data noise

4.5.4.1 Mechanical problems of prototype

It was tried to cover the mechanical problems of the second prototype in the third prototype. It should be noted that these are **prototypes** and not the final *apparatus*. “A **prototype** is an early sample, model of a product built to test a concept or process or to act as a thing to be replicated or learned from ... A prototype is designed to test and try a new design to enhance precision by system analysts and users. Prototyping serves to provide specifications for a real, working system rather than a theoretical one” (Wikipedia contributors 2016). Hence it is not strange to observe data noise. Figure 4-24 shows a meaningful path of design evolution to reach the final design. It was designed to cover the problems of the second prototype in the third prototype. It should be noted that the *apparatus* is often the final design of prototype after solving the problems. The third prototype is a good candidate after some improvements to be commercialised as the soil plasticity determination apparatus.



Figure 4-24: Extract from Wikipedia contributors (2016). An array of prototypes leading to the final design

4.5.4.2 Simplification of test method

In the current study, each data point is derived from only one trial of soil extrusion. However, each data point from the standard method is derived from two or more test trials. For instance, in the case of liquid limit determination by fall cone method: “4.3.3.8 If the difference between the first and second penetration readings is not more than 0.5 mm record the average of the two penetrations and proceed to 4.3.3.9. If the second penetration is more than 0.5 mm and less than 1 mm different from the first, carry out a third test. If the overall range is then not more than 1 mm record the average of the three

penetrations and proceed to 4.3.3.9. If the overall range is more than 1 mm remove the soil from the cup, remix and repeat 4.3.3.3 to 4.3.3.8 until consistent results are obtained and then proceed to 4.3.3.9.” (BS EN 1377-2 1990). Hence, each data point according to fall cone method would be obtained by at least 2 trials and maybe 6 trials or more for some problematic soils as it was experienced in this investigation.

In the case of plastic limit determination by the standard method each data point is also obtained by two or more trials: “Calculate the moisture content of both samples by testing in accordance with the method described in 3.2 but utilizing the samples obtained from 5.3.3. If the two results differ by more than 0.5 % moisture content, repeat the whole test. Calculate the average of the two moisture content values and express the value to the nearest whole number. This is the plastic limit” (BS EN 1377-2 1990). Furthermore, inaccuracy of water content determinations by standard methods also affects the results obtained by proposed method. In addition, the soil extrusion time has been recorded manually by a stopwatch. This makes some noise especially in quick extrusions (i.e. less than 0.5 s). It should be noted that capability of proposed method for determination of soil consistency states with only one trial for each data point is among the advantages of proposed method to current standard methods.

4.5.4.3 Inaccuracy of standard test methods for determination of soil plasticity

As it was discussed earlier, the standard methods are not accurate themselves and provide data with some uncertainty. Even in the determination of one particular limit state, there are very different results by different methods.

Inaccuracy in determination of liquid limit by standard methods

Prakash and Sridharan (2006) provided some data from the literature to have a comparison between liquid limits obtained from the fall cone method and Casagrande percussion cup method (Figure 4-25).

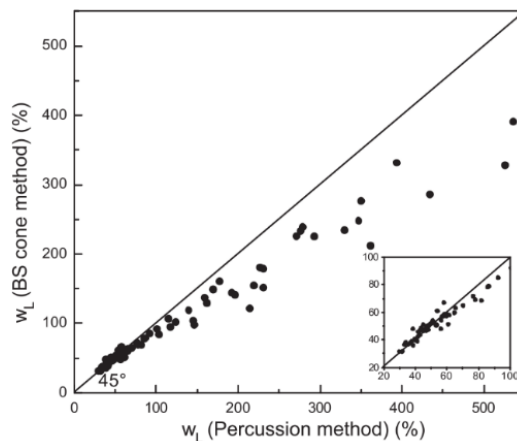


Figure 4-25: Extract from Prakash and Sridharan (2006). Comparison between liquid limits obtained from the fall cone method and Casagrande percussion cup method

The data under 100% water contents for different liquid limits obtained by different methods have been analysed to find the boundary of acceptable liquid limits. The data extracted from Figure 4-25 and the pertinent statistical analysis are presented in Table 4-26 to Table 4-28 and Figure 4-26. The data are grouped into three intervals to have a more realistic analysis. As it was discussed the preference of each method for determination of liquid limits of soils in chapter 2, it is necessary to consider the results of both of the methods for sensitivity analysis of prototype. Utilising four sigma strategy, it is possible to estimate the upper and lower boundaries of acceptable liquid limit determination by standard methods. According to this method, about 95% of possible data fall inside the interval with a distance of ± 2 standard deviation from the mean value of the data. It is also possible to use six sigma strategy to cover more than 99% of possible data inside the boundaries. However, it will lead to wider boundaries that might seem unlikely for some soils. Consequently, the upper and lower boundaries of liquid limit determination by standard methods are shown in Figure 4-27.

Table 4-26: Statistical analysis on LL determination by different standard methods for 25-50%

Casagrande Method ¹	Fall Cone Method ¹	Average Liquid Limit ²	(Casagrande - Fall cone) ³	(Casagrande - linear Casagrande) ⁴	(Fall cone - linear fall cone) ⁵	Casagrande upper bound ⁶	Casagrande lower bound ⁷	Fall cone upper bound ⁸	Fall cone lower bound ⁹
29.52%	31.88%	30.70%	-2.35%	-2.37%	-1.55%	35.2%	28.6%	36.8%	30.1%
31.49%	31.88%	31.68%	-0.38%	-1.20%	-2.45%	36.0%	29.4%	37.7%	31.0%
32.91%	36.75%	34.83%	-3.84%	-0.58%	1.54%	36.8%	30.2%	38.6%	31.9%
33.57%	38.75%	36.16%	-5.18%	-0.72%	2.65%	37.6%	31.0%	39.4%	32.8%
35.32%	37.00%	36.16%	-1.68%	0.23%	0.01%	38.4%	31.8%	40.3%	33.7%
38.50%	36.25%	37.37%	2.25%	2.61%	-1.64%	39.2%	32.6%	41.2%	34.5%
36.63%	38.50%	37.57%	-1.87%	-0.06%	-0.28%	40.0%	33.4%	42.1%	35.4%
37.40%	39.38%	38.39%	-1.97%	-0.09%	-0.29%	40.8%	34.2%	43.0%	36.3%
39.48%	40.75%	40.12%	-1.27%	1.19%	0.19%	41.6%	35.0%	43.9%	37.2%
41.34%	39.38%	40.36%	1.97%	2.25%	-2.08%	42.4%	35.8%	44.8%	38.1%
41.56%	42.50%	42.03%	-0.94%	1.67%	0.16%	43.2%	36.6%	45.7%	39.0%
42.65%	43.88%	43.26%	-1.22%	1.96%	0.64%	44.0%	37.4%	46.6%	39.9%
38.17%	48.38%	43.27%	-10.21%	-3.32%	4.25%	44.8%	38.2%	47.5%	40.8%
41.67%	46.13%	43.90%	-4.46%	-0.62%	1.11%	45.6%	39.0%	48.4%	41.7%
43.09%	47.63%	45.36%	-4.53%	0.00%	1.71%	46.4%	39.8%	49.3%	42.6%
44.95%	46.75%	45.85%	-1.80%	1.06%	-0.05%	47.2%	40.6%	50.1%	43.5%
43.64%	49.25%	46.44%	-5.61%	-1.05%	1.56%	48.0%	41.4%	51.0%	44.4%
46.70%	47.13%	46.91%	-0.42%	1.21%	-1.46%	48.8%	42.2%	51.9%	45.2%
48.34%	47.63%	47.98%	0.72%	2.05%	-1.85%	49.6%	43.0%	52.8%	46.1%
46.81%	49.38%	48.09%	-2.56%	-0.28%	-0.99%	50.4%	43.8%	53.7%	47.0%
44.84%	51.50%	48.17%	-6.66%	-3.05%	0.24%	51.2%	44.6%	54.6%	47.9%
47.80%	50.75%	49.27%	-2.95%	-0.90%	-1.40%	52.0%	45.4%	55.5%	48.8%
Count			22	22	22				
Mean			-2.50%	0.00%	0.00%				
Max. Difference			2.25%	2.61%	4.25%				
Min. Difference			-10.21%	-3.32%	-2.45%				
Standard Deviation (SD)			2.87%	1.65%	1.67%				

1- Extracted LL data from Figure 4-25

2- Average LL value of Casagrande and fall cone methods

3- (Casagrande LL value – fall cone LL value)

4- (Casagrande LL value – Casagrande linear regression value)

5- (Fall cone LL value – fall cone linear regression value)

6- Casagrande upper bound value based on 4 σ method

7- Casagrande lower bound value based on 4 σ method

8- Fall cone upper bound value based on 4 σ method

9- Fall cone lower bound value based on 4 σ method

Table 4-27: Statistical analysis on LL determination by different standard methods for 50-75%

Casagrande Method ¹	Fall Cone Method ¹	Average Liquid Limit ²	(Casagrande - Fall cone) ³	(Casagrande - linear Casagrande) ⁴	(Fall cone - linear fall cone) ⁵	Casagrande upper bound ⁶	Casagrande lower bound ⁷	Fall cone upper bound ⁸	Fall cone lower bound ⁹
49.33%	52.13%	50.73%	-2.80%	-1.75%	2.12%	56.8%	45.3%	54.7%	45.3%
53.16%	51.00%	52.08%	2.16%	1.11%	0.00%	57.8%	46.3%	55.7%	46.3%
55.90%	48.38%	52.14%	7.52%	2.88%	-3.62%	58.8%	47.3%	56.7%	47.3%
50.86%	54.38%	52.62%	-3.51%	-3.12%	1.39%	59.7%	48.2%	57.7%	48.3%
56.11%	54.50%	55.31%	1.61%	1.16%	0.52%	60.7%	49.2%	58.7%	49.3%
60.60%	51.63%	56.11%	8.98%	4.68%	-3.35%	61.7%	50.2%	59.7%	50.3%
56.77%	57.50%	57.14%	-0.73%	-0.11%	1.52%	62.6%	51.1%	60.7%	51.2%
53.60%	61.25%	57.42%	-7.65%	-4.26%	4.28%	63.6%	52.1%	61.7%	52.2%
59.51%	57.13%	58.32%	2.38%	0.69%	-0.84%	64.6%	53.1%	62.7%	53.2%
58.30%	58.63%	58.46%	-0.32%	-1.49%	-0.34%	65.5%	54.0%	63.7%	54.2%
59.95%	59.00%	59.47%	0.95%	-0.81%	-0.96%	66.5%	55.0%	64.7%	55.2%
61.59%	58.25%	59.92%	3.34%	-0.14%	-2.70%	67.5%	56.0%	65.7%	56.2%
64.65%	59.88%	62.26%	4.78%	1.96%	-2.07%	68.5%	56.9%	66.7%	57.2%
57.87%	67.25%	62.56%	-9.38%	-5.79%	4.31%	69.4%	57.9%	67.7%	58.2%
65.20%	63.50%	64.35%	1.70%	0.57%	-0.44%	70.4%	58.9%	68.7%	59.2%
70.01%	65.13%	67.57%	4.89%	4.42%	0.19%	71.4%	59.8%	69.7%	60.2%
Count			16	16	16				
Mean			0.87%	0.00%	0.00%				
Max. Difference			8.98%	4.68%	4.31%				
Min. Difference			-9.38%	-5.79%	-3.62%				
Standard Deviation (SD)			4.95%	2.88%	2.36%				

1- Extracted LL data from Figure 4-25

2- Average LL value of Casagrande and fall cone methods

3- (Casagrande LL value – fall cone LL value)

4- (Casagrande LL value – Casagrande linear regression value)

5- (Fall cone LL value – fall cone linear regression value)

6- Casagrande upper bound value based on 4σ method

7- Casagrande lower bound value based on 4σ method

8- Fall cone upper bound value based on 4σ method

9- Fall cone lower bound value based on 4σ method

Table 4-28: Statistical analysis on LL determination by different standard methods for 75-100%

Casagrande Method ¹	Fall Cone Method ¹	Average Liquid Limit ²	(Casagrande - Fall cone) ³	(Casagrande - linear Casagrande) ⁴	(Fall cone - linear fall cone) ⁵	Casagrande upper bound ⁶	Casagrande lower bound ⁷	Fall cone upper bound ⁸	Fall cone lower bound ⁹
77.56%	69.38%	73.47%	8.19%	2.97%	2.77%	79.2%	70.0%	72.6%	60.7%
76.47%	72.00%	74.24%	4.47%	-1.84%	1.60%	82.9%	73.7%	76.3%	64.5%
81.61%	68.88%	75.24%	12.74%	-0.40%	-5.33%	86.6%	77.4%	80.1%	68.3%
85.66%	78.50%	82.08%	7.16%	-0.06%	0.50%	90.3%	81.1%	83.9%	72.1%
86.43%	79.50%	82.96%	6.93%	-3.01%	-2.30%	94.0%	84.8%	87.7%	75.9%
92.34%	85.38%	88.86%	6.96%	-0.81%	-0.22%	97.8%	88.5%	91.5%	79.7%
100.00%	92.38%	96.19%	7.63%	3.15%	2.98%	101.5%	92.2%	95.3%	83.4%
Count			7	7	7				
Mean			7.73%	0.00%	0.00%				
Max. Difference			12.74%	3.15%	2.98%				
Min. Difference			4.47%	-3.01%	-5.33%				
Standard Deviation (SD)			2.50%	2.31%	2.97%				

1- Extracted LL data from Figure 4-25

2- Average LL value of Casagrande and fall cone methods

3- (Casagrande LL value – fall cone LL value)

4- (Casagrande LL value – Casagrande linear regression value)

5- (Fall cone LL value – fall cone linear regression value)

6- Casagrande upper bound value based on 4σ method

7- Casagrande lower bound value based on 4σ method

8- Fall cone upper bound value based on 4σ method

9- Fall cone lower bound value based on 4σ method

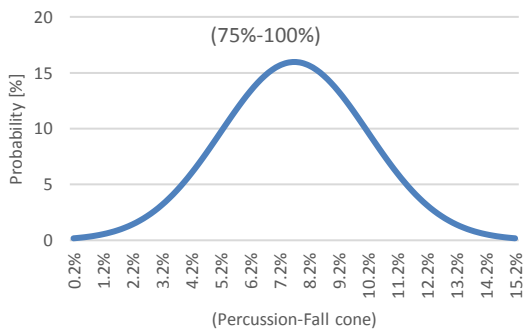
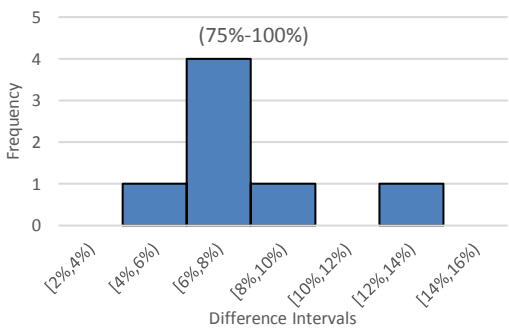
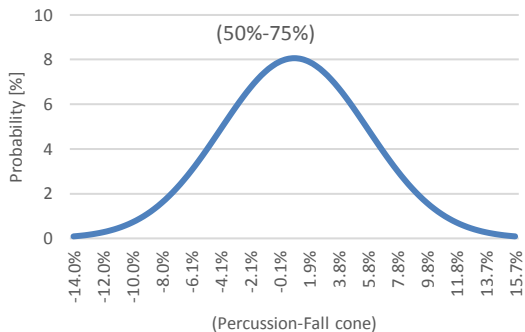
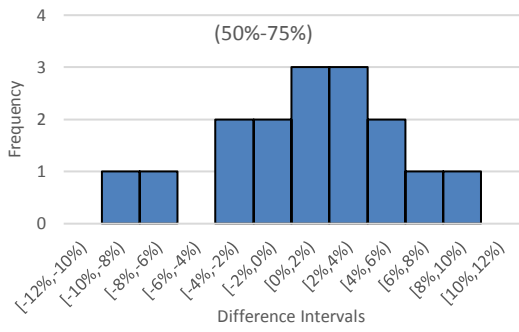
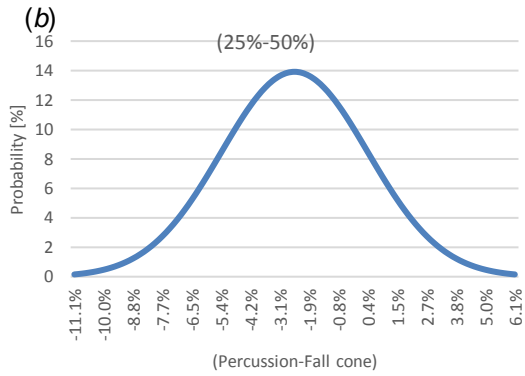
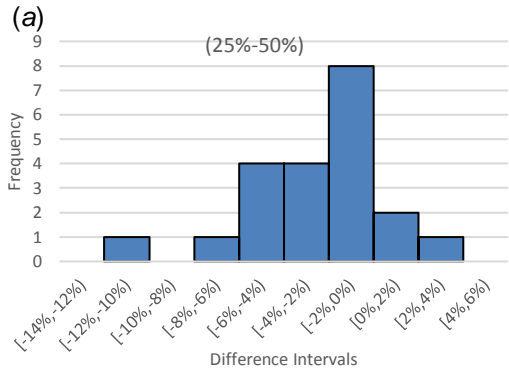


Figure 4-26: Statistical analysis of difference between results of standard methods for determination of liquid limit (Percussion Method - Fall Cone Method): a) Histograms for different intervals; b) Normal distribution curve for different intervals

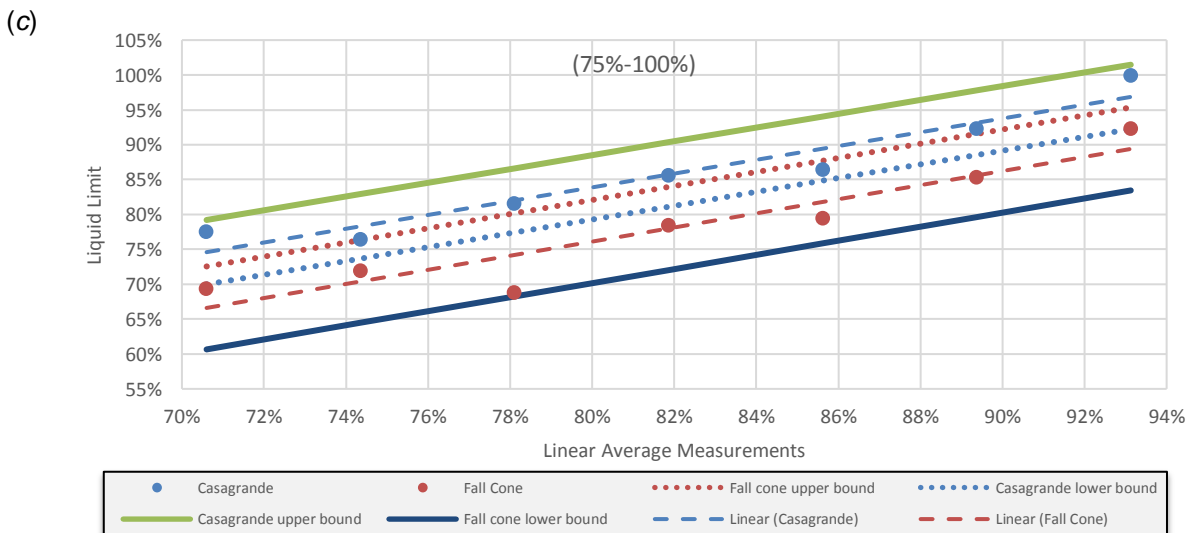
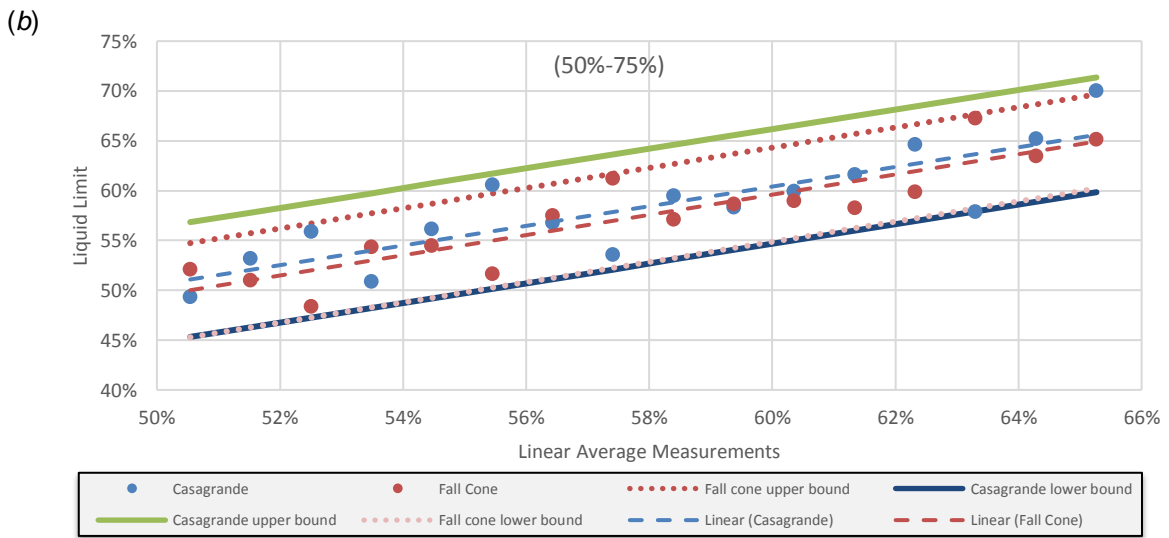
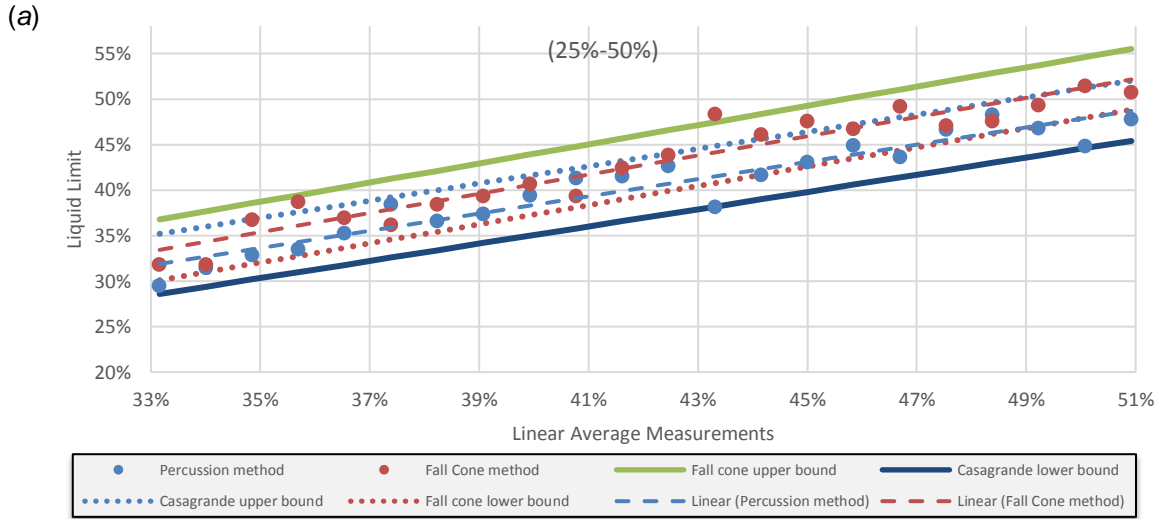


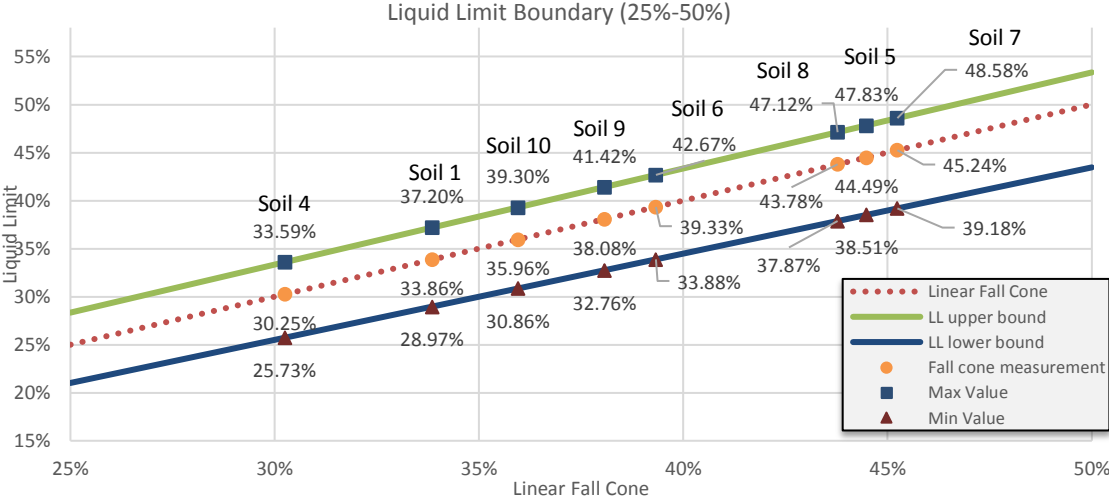
Figure 4-27: Comparison between Percussion and Fall Cone Methods in addition to determination of liquid limit boundary based on 4σ method: a) 25%-50%; b) 50%-75%; c) 75%-100%

According to the graphs, for soils with liquid limits between 25% and 50%, fall cone method usually gives higher values than percussion cup method. The liquid limit interval of 50% to 75% is a transitional domain that percussion cup method values exceed the fall cone method values and this trend continues.

Minimum and Maximum liquid limit determination

Based on the statistical analysis of different results obtained by two different standard methods of liquid limit determination, it is possible to find the minimum and maximum liquid limit values as they are shown in Figure 4-28.

(a)



(b)

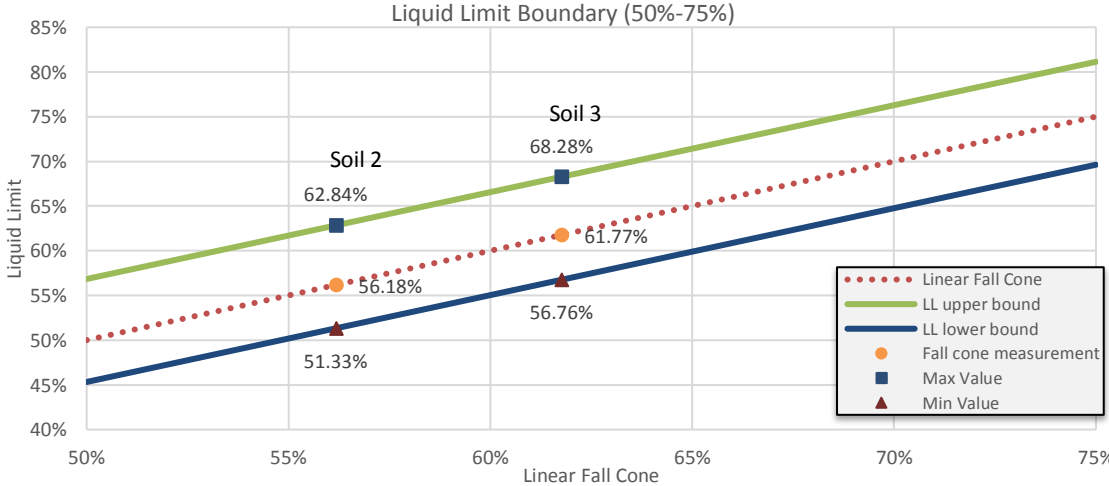


Figure 4-28: Determination of liquid limit boundary for soils: a) 25%-50%; b) 50%-75%

Inaccuracy in determination of plastic limit by standard method

As it had been discussed the difficulties in determination of plastic limit by the standard method, plastic limits of soils are determined differently by various operators and even by the same operator with different soil types. Medhat and Whyte (1986) investigated

the reproducibility of plastic limit test on four different soils with several operators and showed a lack of reproducibility for the test as are shown in Figure 4-29. Normal distribution curves of the data are presented in Figure 4-30. Variability in plastic limit determination for soils based on the 4σ method and the borders of plastic limit determination are shown in Figure 4-31.

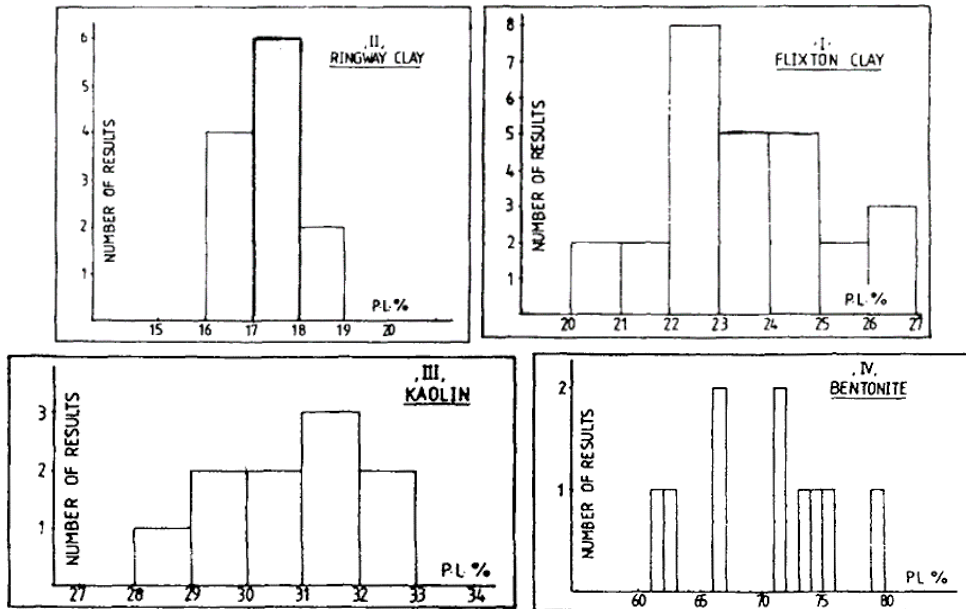


Figure 4-29: Extract from Medhat and Whyte (1986). Plastic limit test results by different operators

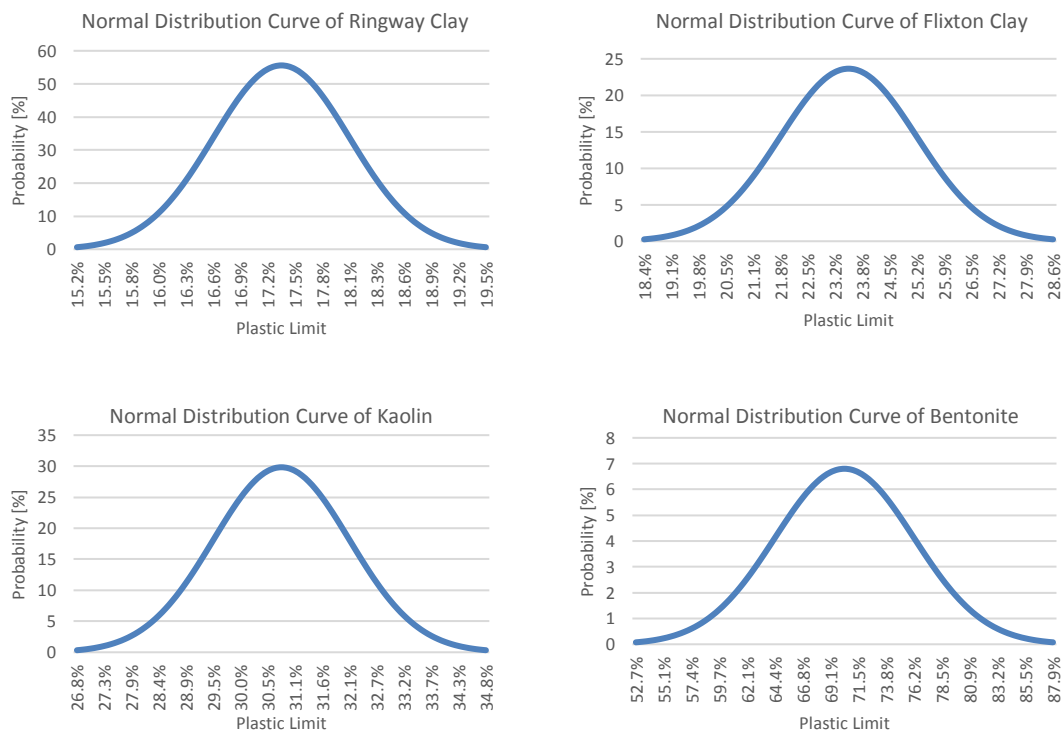


Figure 4-30: Normal distribution curves of the data extracted from Figure 4-29

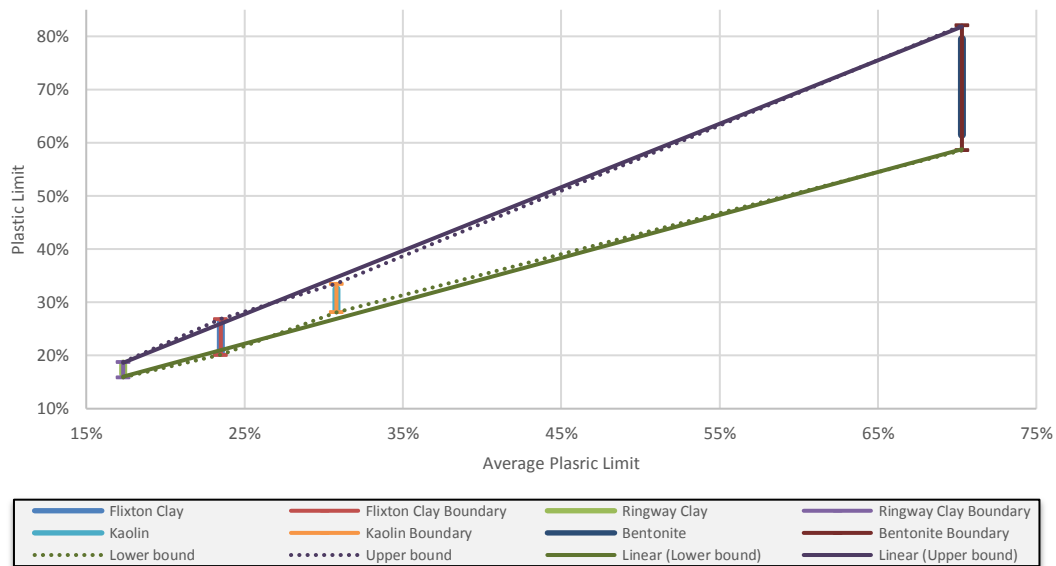


Figure 4-31: Variability in plastic limit determination for soils studied by Medhat and Whyte (1986) based on 4σ method and plastic limit determination borders

As the Figure 4-29 and Figure 4-31 show, the variability of plastic limit determination increases by the rise in plastic limit value. This shows that operator's performance and judgment have more influence on the plastic limit determination for soils with higher *PL* values.

Minimum and Maximum plastic limit determination

Based on the statistical analysis of variability in plastic limit determination by different operators for the soils studied by Medhat and Whyte (1986), it is possible to find the minimum and maximum plastic limit values of the soils investigated in current research. As it is known, every obtainable data from the standard methods is only one point within the acceptable range shown in Figure 4-31. For example, for soil 1, as shown in Figure 4-32, the plastic limit of soil is 19.76% based on standard methods. This point might be anywhere between the upper boundary and the lower boundary. Hence, the point might even be on the borders. Therefore, *PL* of the soil could be between [16.79, 19.76] or [19.76, 24.15]. There are two ways to specify the position of the point more accurately. 1) the author should have participated in the research of Medhat and Whyte in 1986 to find out the position of his determinations relative to the others (which is impossible). 2) The author himself should do another research like what Medhat and Whyte did (which is impractical considering research budget and time, however, the author intends to do it in the future studies). Considering the restrictions, the author assumed that his determination would be close to the average point of possible determinations for that specific soil. Hence *PL*, of the soil could be between [19.79, 21.53] as it is shown in Figure 4-32. Acceptable ranges of plastic limit determination of soils by the current standard method are also shown in Figure 4-33.

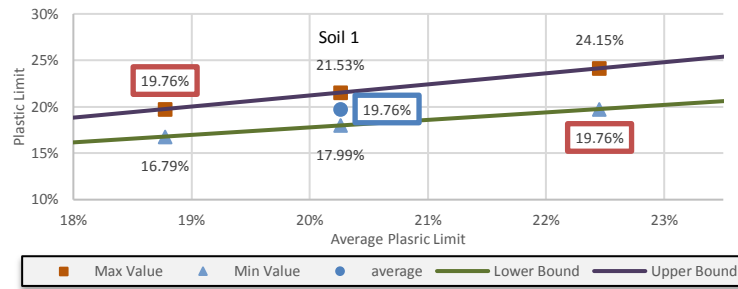


Figure 4-32: Determination of plastic limit boundary for soil 1

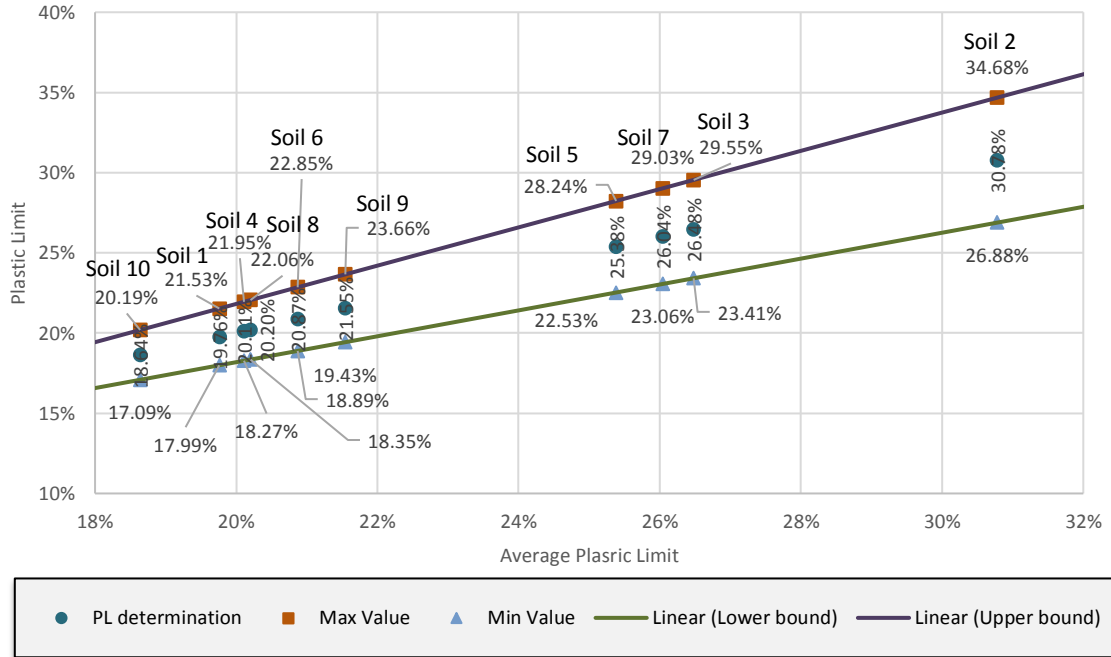


Figure 4-33: Variability in plastic limit determination of soils by current standard method

4.5.4.4 Inaccuracy in water content determination

During the experimental works, the author noticed that the digital balance available in the laboratory used for determination of water contents had an error with ± 0.04 g, which is more than 0.01 g resolution for these type of electronic medium top-pan balances (Head 2006)). Although this small fraction does not have a great effect on the final results, it has some effects on R^2 values in graphs. The author relates this error to sensitivity of balance to humidity and temperature of room while testing. The author could reduce this error by keeping the balance on during the test days.

4.5.5 Variability of plasticity determination of experimented soils

According to the previous sections, soil plasticity determination does not provide a unique value, but a wide range of values. The statistical limit state boundaries determination in previous sections confirms Terzaghi et al. (1996) statement regarding limit states boundaries: "the transition from one state to another does not occur abruptly as soon as some critical water content is reached. It occurs gradually **over a fairly large**

range in the value of the water content. For this reason every attempt to establish criteria for the boundaries between the limits of consistency involves some arbitrary elements” (author’s underlining). These variabilities for investigated soils are shown in Table 4-29 and Figure 4-34. Considering the variability of Atterberg limit tests, the soils positions change in plasticity chart too. This will lead to different soil classifications based on possible positions of soil in the chart. Figure 4-35 shows acceptable areas for all soil in this investigation based on current standard methods.

Table 4-29: Atterberg limits variability

Soil No.	1	2	3	4	5	6	7	8	9	10
Standard LL	33.86%	56.18%	61.77%	30.25%	44.49%	39.33%	45.24%	43.78%	38.08%	35.96%
Standard PL	19.76%	30.78%	26.48%	20.11%	25.38%	20.87%	26.04%	20.20%	21.55%	18.64%
Min LL	28.97%	51.33%	56.76%	25.73%	38.51%	33.88%	39.18%	37.87%	32.76%	30.86%
Max LL	37.20%	62.84%	68.28%	33.59%	47.83%	42.67%	48.58%	47.12%	41.42%	39.30%
Min PL	17.99%	26.88%	23.41%	18.27%	22.53%	18.89%	23.06%	18.35%	19.43%	17.09%
Max PL	21.53%	34.68%	29.55%	21.95%	28.24%	22.85%	29.03%	22.06%	23.66%	20.19%
Plasticity Index	14.10%	25.40%	35.29%	10.14%	19.11%	18.46%	19.20%	23.57%	16.54%	17.32%
Min LL - Min PL	10.98%	24.44%	33.35%	7.46%	15.99%	14.99%	16.12%	19.52%	13.33%	13.77%
Max LL - Max PL	15.67%	28.17%	38.73%	11.64%	19.60%	19.82%	19.55%	25.06%	17.76%	19.11%
Min PI	7.44%	16.65%	27.21%	3.78%	10.28%	11.03%	10.15%	15.81%	9.10%	10.67%
Max PI	19.21%	35.96%	44.86%	15.32%	25.31%	23.78%	25.52%	28.77%	21.99%	22.21%

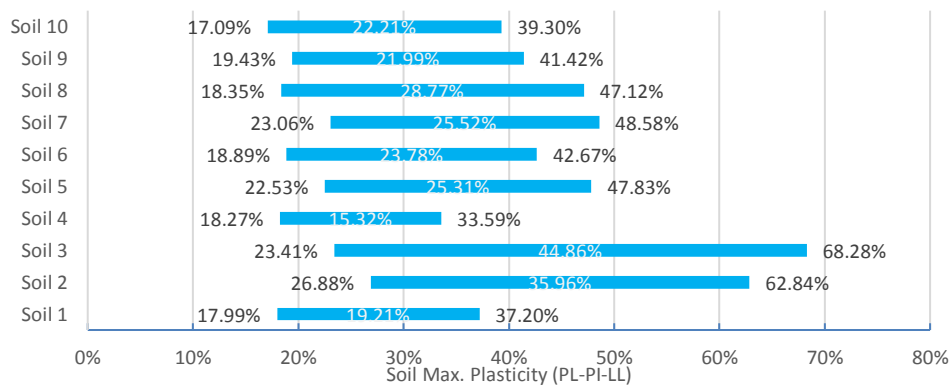


Figure 4-34: Soils plasticity comparison in case of maximum PI

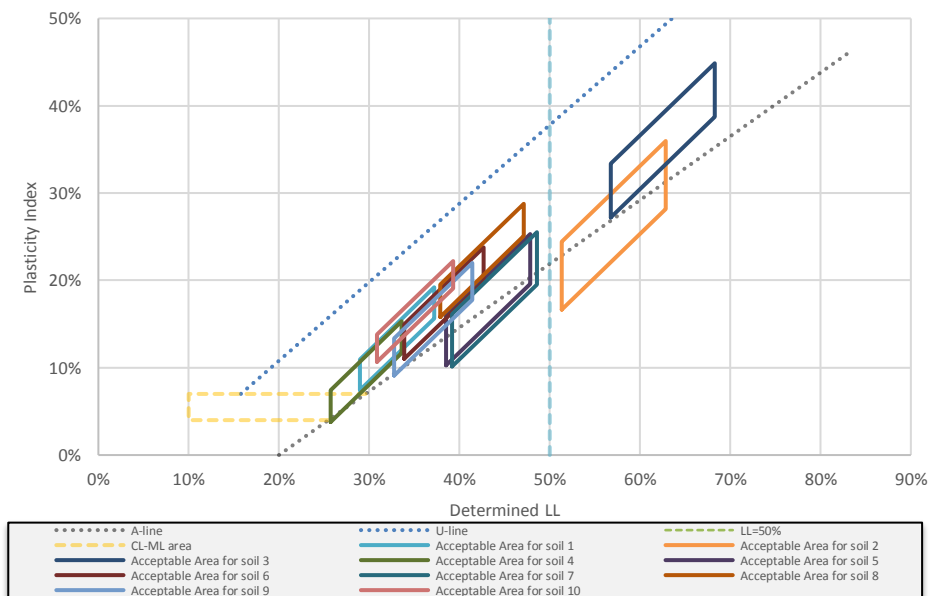


Figure 4-35: Soils position variability in plasticity chart

Variability in soil plasticity determination will lead to classifying soils into different categories, which shows the inaccuracy of determination of soil plasticity by current standard methods. Different possible soils classifications are presented in Table 4-30. According to this table, the soils selected for this investigation can be considered as representative of all possible inorganic soil types.

Table 4-30: Possible soil type classification according to plasticity chart

Soil Sample	1	2	3	4	5	6	7	8	9	10
Possible Soil Types	CL	MH	CH	CL	CL	CL	CL	CL	CL	CL
	-	CH	-	CL-ML	ML	-	ML	-	ML	-
	-	-	-	ML	-	-	-	-	-	-

4.5.6 Effect of soil plasticity determination variability on soil behaviour graphs in proposed method

Considering Atterberg limits determination variability by current standard methods, data values also vary in Time – *L* graphs of proposed method as it is shown in Figure 4-36. This graph shows that the difference between the results obtained by the prototype and current standard method in Table 4-19 can also be due to the inaccuracy of current standard methods because the generalised path can be drawn anywhere between specified borders. Although these borders will be significantly limited by following the standard procedure exactly and by a particular operator, following current standard procedures for different soil types might even be difficult for a particular operator especially for determination of plastic limits. Hence, it is important to consider the third proposed approach discussed earlier for calibration of the proposed apparatus.

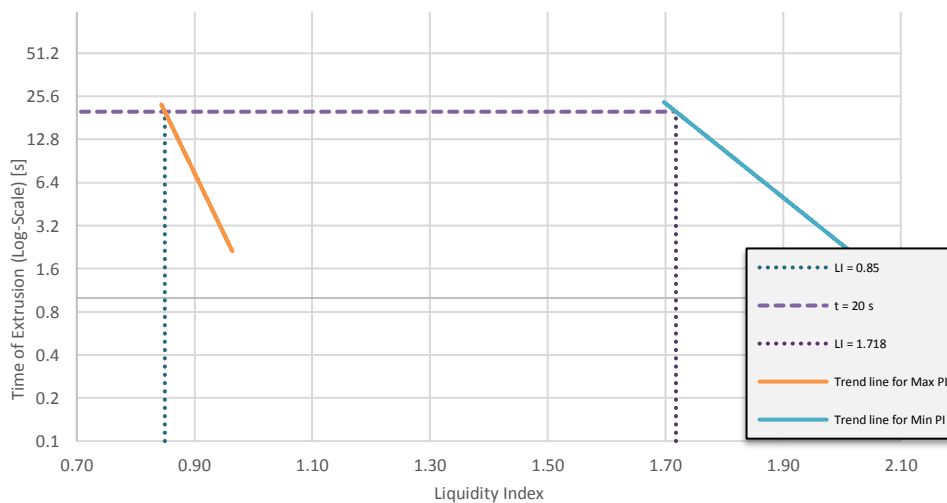


Figure 4-36: Possible borders of soil 1 behaviour data under about 34.8 kPa

5 Conclusions and Recommendations

Conclusions derived from the conducted research on soil plasticity determination are presented in this chapter. In addition, recommendations for future works are suggested accordingly.

5.1 Conclusions

Soil consistency is a term that expresses the level of firmness of cohesive soils which is a qualitative phenomenon. It depends on the behaviour of soil while its moisture content varies. Many different parameters, such as soil texture and structure, soil grain size and shape, environmental condition of soil, soil morphology and many other factors, affect the behaviour of soil. The problem initiates from the point of quantifying a qualitative phenomenon. The problem becomes more serious when only one parameter, like soil shear strength, is measured and becomes an indication for determination of a complex system.

There are many issues and problems related to the conventional methods and apparatus to determine liquid and plastic limits of soils that directly and indirectly influence the results of tests. There have been many studies to eliminate the errors and uncertainties pertinent to the regular established methods of Atterberg limits determination. Most of them provide a correlation between the results of their apparatus to the results of standard methods for determination of Atterberg limits or to a specific parameter of soil (e.g. shear strength). In addition, there is not a clear and precise definition of soil plasticity in the literature. It is important to note that the procedure of different methods are not definitions, but are indications for reaching to a particular state of soil. None of the laboratory tests are perfect and their results should not be considered as a certain benchmark for future investigations. Unfortunately, many researches have been done to correlate their results with the values obtained by current standard methods in condition that we know the results obtained by current standard methods are not reliable necessarily.

Soil consistency is a qualitative phenomenon and it is required to observe the behaviour of soil when its water content varies. In this regard, author's suggestion for solving the problems related to the determination of soil plasticity is utilisation of qualitative research approach. It is required to do a qualitative research that covers most important factors then quantifying the research results. In this regard, a new test method and an apparatus for determination of soil consistency based on the nature of soil deformation were proposed, referred to as Manafi method and apparatus. In this technique, it is possible

to determine the workability of soil at Atterberg limits involving affecting parameters on soil behaviour and specifying the consistency states of the soil. In the proposed method, defined systems of soil deformation are correlated to specific soil consistencies (liquid limit and plastic limit states of soil). Hence, the qualitative soil deformation process affected by the most important parameters in soil consistency can be quantified by a simple measurement of deformation time.

The final prototype obtained after fabrication of several prototypes. The apparatus is a direct extrusion apparatus that deforms the soil specimen during the test in a particular condition and lets the operator to measure the deformation rate of soil. Hence, it is possible to quantify the qualitative process of soil deformation by measuring the power of soil deformation.

Ten different soils were selected carefully to cover a wide range of soils plasticity with plastic limits range from 18.64% to 30.78%, and the liquid limits range from 30.25% to 61.77%. The most differences between the results obtained by proposed method and standard methods were 2.38% and 1.94% of water content for liquid and plastic limits correspondingly, which are negligible in comparison to the accuracy of current standard methods.

Having a comparison between coefficients of multiple determination (R^2 values) of results of the proposed method and the standard methods within the soil plasticity range showed that the R^2 values of proposed method in both of limit state determinations are more than standard methods (rise from 0.9001 to 0.9128 for liquid limit determination and from 0.8623 to 0.902 for plastic limit determination). Hence, the proposed method provides more consistent results compare to standard methods (fall cone and thread rolling methods).

Statistical analysis on limit states determination by standard methods confirmed Terzaghi et al. (1996) statement regarding large limit states boundaries for the transition from one state to another state. These large boundaries are not only due to intrinsic apparatus deficiencies/limitations, and operator's performance/judgement, but also due to different mechanisms of the methods. The effort of the current research was for providing a more reliable method for soil plasticity determination based on the concepts of soil consistency determination by qualitative research approach and narrowing the limit states boundaries. The experimental results obtained by designed apparatus confirmed the proposed method and provided more consistent results in comparison to current standard methods.

5.2 Recommendations for future works

The conducted research project was a research base for later comprehensive study covering all deficiencies pertinent to the determination of soil consistency limits. In the comprehensive study, it is required to investigate the combination effect of various parameters such as full particle sized sampling, sampling destruction, absorbed water determination, various chemistry and pH of water used in soil sampling, and environmental conditions on soil consistency determination.

It is recommended that further tests are done on various soil types with higher plasticity range by proposed and standard methods to have a more comprehensive comparison between the methods. In addition, plasticity determination of organic soils by current standard methods is in disputation. The proposed plasticity determination method has a great potential to be a solution for the problem that requires a separate research.

It is also suggested to study the effect of additives, such as chemical additions used in geotechnical soil improvement projects, on soil consistency determination by different methods.

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