MSc in Railway Systems Engineering and Integration

College of Engineering, School of Civil Engineering

University of Birmingham

Feasibility Study of Proposed Sidings Development at Midsomer Norton Station

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Executive Summary

It is planned to reopen Midsomer Norton Railway Station, which is a station on the Somerset & Dorset Heritage Railway Line, to traffic. As a first step, Midsomer Norton will be terminal and it is proposed to build a siding and a depot for the terminating trains.

However, a ground investigation is essential before starting building the siding and the depot, as it is the vital part of any civil engineering process. Ground's behaviour must be known in order to form a structure on it. Data from ground investigation is a reference for any kind of structures.

This study covers a site investigation including a desk study, walkover survey, sampling and laboratory testing in order to perform a suitable design. Moisture content test, liquid limit and plastic limit tests, standard compaction test, shear vane test and triaxial compression tests were carried out to determine the soil characteristics and to analyse the soil behaviour.

4 trial pits were excavated to depth of 0.8 metres, layers and visual properties of the soil were identified on site. The material is identified as mudstone of Mercia Mudstone Group, which is commonly encountered in constructions in the UK.

Natural moisture content of the soil was obtained from moisture content tests applied on 8 samples, ranges from 27.98% to 40.57% depending on the location of the samples. Similarly, Atterberg Limits tests were performed on 8 samples and liquid limit was found between 63.66% and 90.13%, and plastic limit is between 37.92% and 54.67%. On the basis of these values, the soil material is defined as very high plasticity clay.

After classification, compaction and shear strength tests were carried out and they gave the results of average optimum moisture content is 33.55%, maximum dry density is 1.34 g/cm^3; cohesion c' = 75 Kpa and shearing resistance angle $\phi = 41°$.

Results of the tests were evaluated and slope stability and bearing capacity analysis were processed. An alignment was drawn using Autocad and the amount of earthwork was calculated. It was found that the ground is capable to build the proposed structure, however there is a need for 370.123 m^3 fill earthwork.

A strong recommendation was made to follow systems engineering approach in the construction phase, as well as the design stage. Dependability of the system is ensured by the good use of V – process of systems engineering.

It is also recommended to undertake further testing including sulphate content and consolidation tests to confirm the validity of the design processed in this study.

Keywords: Ground investigation, railway siding, embankment, soil strength, earthwork, laboratory testing on soil.

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Glossary of Terms / List of Abbreviations

Term	Explanation / Meaning / Definition
BGS	British Geological Survey
CBR	California Bearing Ratio
LI	Liquidity Index
LL	Liquid Limit
MCV	Moisture Condition Value
MDD	Maximum Dry Density
MM	Mercia Mudstone
OMC	Optimum Moisture Content
PL	Plastic Limit
RAMS	Reliability Availability Maintainability Safety
S&DHRT	Somerset and Dorset Railway Heritage Trust
TPxBx	Trial Pit x Bag x

1 Introduction

Railway transportation is becoming the World's transportation mode day by day. As railways sector is developing, there is a high demand for new research projects, innovative methods on railway constructions. With the engineering skills and experiences, building new railways is much easier. Not only building new railways, but also maintenance and refurbishment of existing railway assets become less effortful with engineering excellence.

Engineering supervision is the key to ensure the best use of an investigation. Sophisticated laboratory techniques and equipment, use of powerful computational methods are of little value if there is no engineering supervision.

1.1 Aims & Objectives

In this study, a detailed site investigation and design of a railway siding and a depot is processed. The investigation aims to understand the ground conditions to see if it is capable to carry a predetermined load that the siding will cause and it guides the design process as the design is produced using the data from the ground investigation.

Objectives of this study include identifying the problem, conducting a ground investigation, and a proper design of the proposed siding and depot.

Ground investigation is the most important part of a civil engineering project, as it decides the feasibility of the proposed construction. All the work is done considering the data from ground investigation.

In this study, a proper desk study, field testing, sampling, lab testing and analysing the ground conditions give a clear idea for the proposed construction of siding, and a useful design is presented.

1.2 Scope

This study is part of a renewal project. Ground investigation and design of the earthworks are included in the scope. However, construction phase needs further study. This study covers re-arranging grade of the ground for siding, embankments and the depot, realigning the footpath, ground investigation by sampling and doing tests, final design of the earthwork and drainage for the siding and the depot.

During this renewal and reopening process, some other issues are out of scope and need further investigation and design. For example, there is a need for building a bridge on the place of the demolished bridge over Silver Street, to connect the line to the northern side, for later stages. Also, there is a rubbish dump on the track in the south of the Midsomer Norton station, which needs to be removed to open the line to traffic.

Design of superstructure of the siding and the depot is not analysed in detail and is not in the scope of this study. Also some new trains are needed to buy to run the line; specifications of the trains are not included in the scope.

1.3 Methodology

Investigation started with desk study. A walk-over survey was performed and soil was classified by description and simple testing. A detailed subsurface exploration and field testing was processed. Samples were taken from the site and were tested in laboratory. After the tests, the data was evaluated and a proper geotechnical design was fulfilled.

1.3.1 Desk Study and Walk-over Survey

A desk study was performed to identify both previous and existing land use in the area of the site. Location of the proposed construction, site geology, and groundwater related features were analysed.

Main purpose of the desk study and walkover survey is to determine the conditions of soil and to identify the probability of unexpected hazards and potential construction problems.

During desk study, required information about the site, necessary pictures, maps, air photographs and documents were investigated and analysed in addition to a walkover survey.

Walkover survey was conducted to check the accuracy of the information obtained in the desk study. Also, some important factors may be realised during the walkover survey, which could not be seen in the desk study. There might be changes on the site since the maps were published.

The first walkover survey was conducted on 27th April 2013 by the author with Peter Russell and John Dora of S&DHRT.

The second walkover survey was conducted on 23rd July 2013 with Dr. Gurmel Ghataora and Peter Russell before sampling.

1.3.2 Subsurface Exploration and Field Testing

The ground under the proposed line of the siding was investigated doing some in-situ tests as well as lab tests.

Trial pits were excavated to take samples. As number of available equipment is limited for in-situ tests, more lab testing is undertaken than in-situ testing.

During the planning and progress of a site investigation, it is essential to decide on the method of test to be used. Because, even a well done test is not useful if the test method is not appropriate.

1.3.3 Sampling and Laboratory Testing

Sampling was carried out in order to perform laboratory testing. There are several types of lab tests to determine the ground conditions. University of Birmingham laboratory is used to carry out lab testing. Enough number of samples was taken to do necessary tests. Sampling was done carefully and each sample was labelled with reference number of hole and sample, date of sampling and depth of top and bottom of the sample.

Soil classification and ground properties were determined by doing lab tests.

1.3.4 Design

Engineering excellence makes a site investigation meaningful. Sophisticated laboratory techniques and equipment, use of powerful computational methods are of little value if there is no engineering supervision. Engineer is the key person to ensure the best use of site investigation.

The data from the tests was used for designing cut and fill areas to form a platform for the new siding and related buildings. A lot of brainstorming, reading and having exchange of ideas with knowledgeable people were done during the design process to ensure a proper design is issued.

1.4 Assignment Structure

This dissertation starts with introduction which introduces the aims, objectives and scope of the study and methodology followed, and some background information about the site of the project follows. In chapter 3, a detailed literature review about site investigation in general, the geology of the area and the material present on site Mercia Mudstone, is performed. Author conducts a systems engineering approach to the study, and presents it in chapter 4. Chapter 5 covers subsurface exploration and sampling, with some photos from the site, taken in the sampling stage, by the author. Laboratory tests are mentioned in chapter 6, including scope and procedures of the tests, and results with the calculations. Design is described in chapter 7, for drainage and earthworks, with calculations and maps drawn. The study is concluded in chapter 8, reviewing findings and making suggestion. Chapter 9 covers the references. Appendices follow references.

2 Background

The original Somerset & Dorset Central Railway ran from Highbridge to Glastonbury and was opened in 1854. It was gradually extended to Evercreech and over the Mendips to Bath. Colliery traffic from Radstock and Midsomer Norton and quarries of the Mendips increased strategic importance of Midsomer Norton, and the station Midsomer Norton of the S&D railway line opened in 1874. However, the line was closed to traffic in 1966. Now it is a heritage railway where train ridings are done to attract people to see the railway. It is planned to reopen to traffic. As a first stage, it is proposed to build a siding and a depot for the terminating trains. It is needed to undertake a site investigation. The location of the Midsomer Norton Railway Station is the south of Bristol, near Bath city. A map of the Somerset and Dorset Railway network and the station plan of the Midsomer Norton are shown in figures 1 and 2. (S&DHRT)

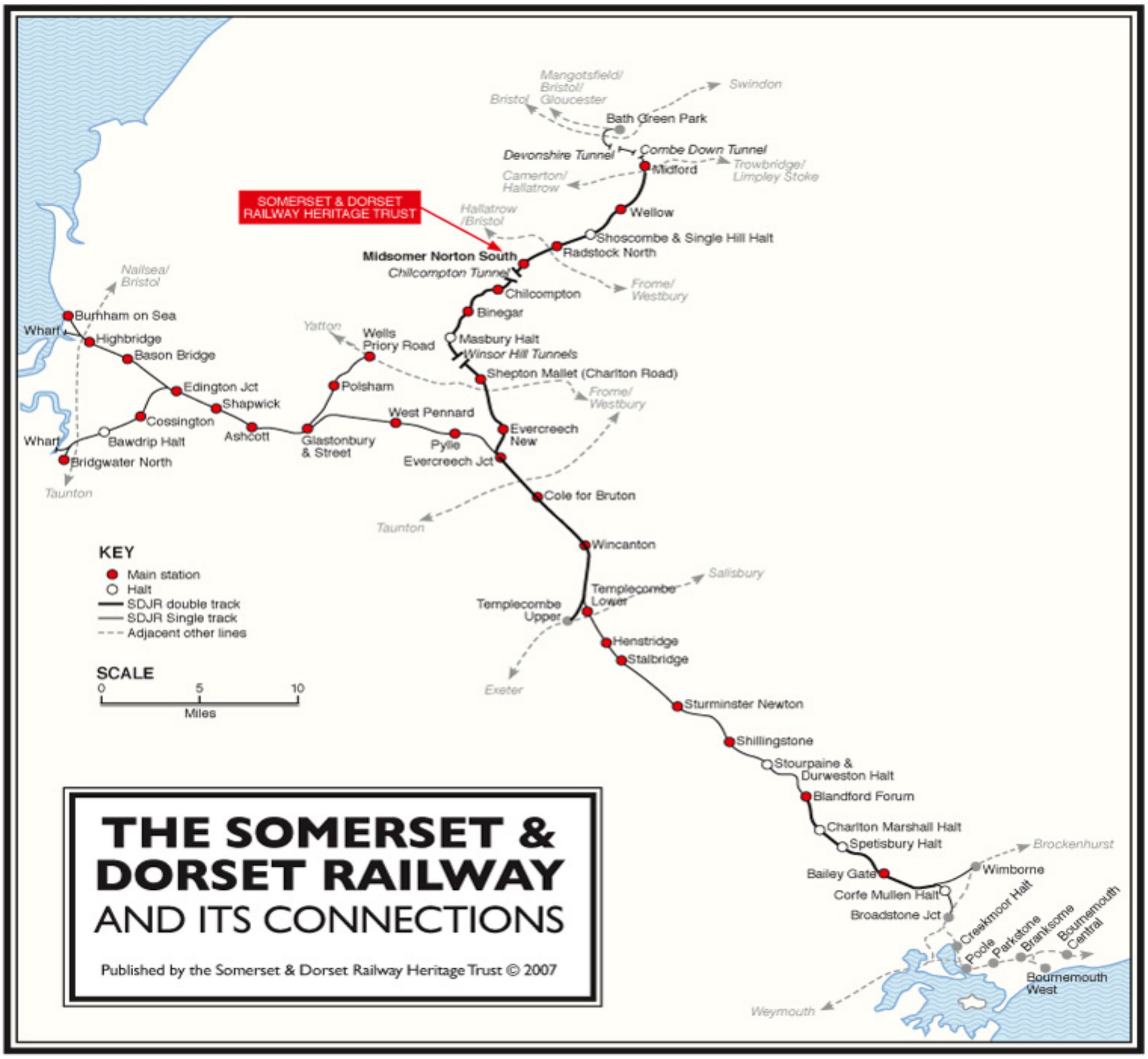


Figure 1 Original S&D Route (S&DHRT)

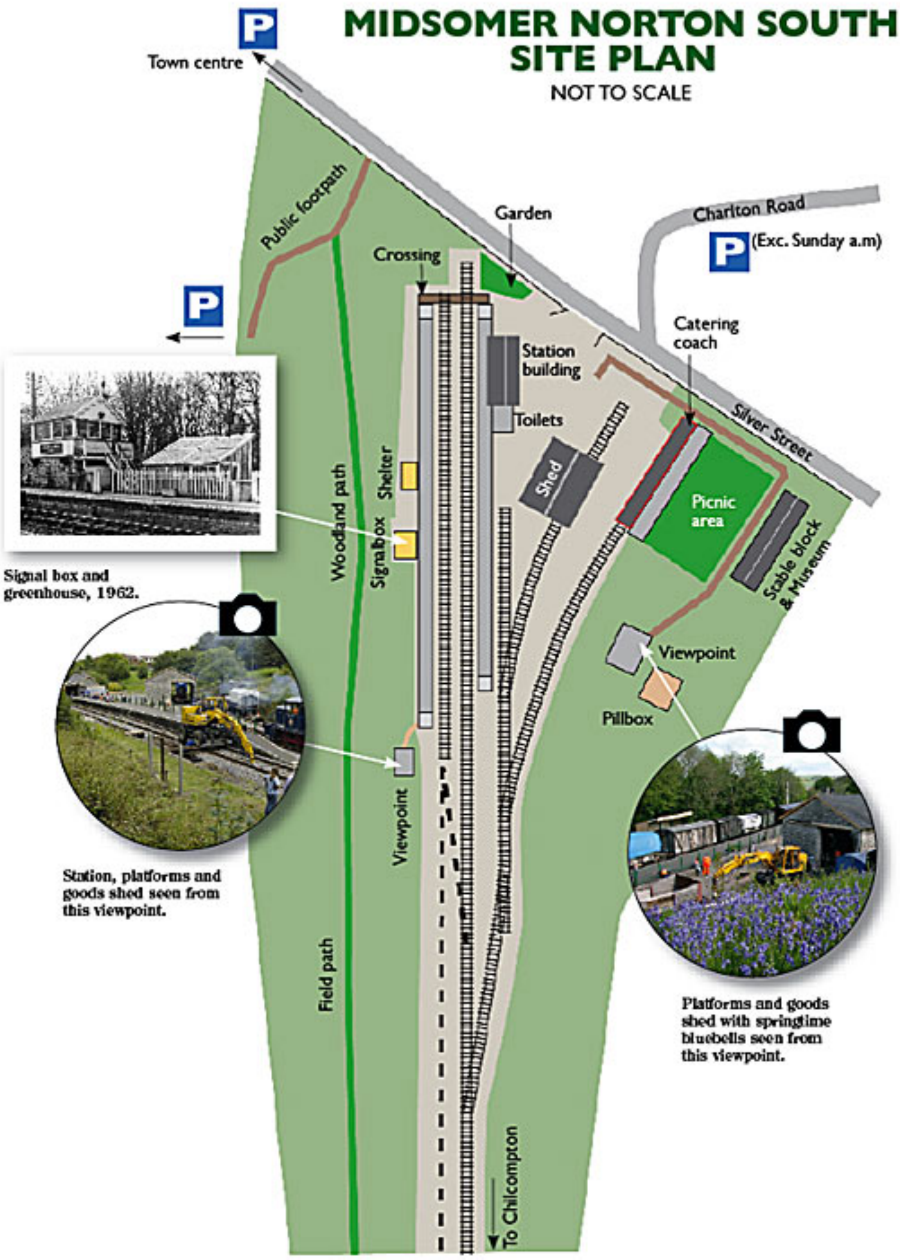


Figure 2 The Station Site (S&DHRT)

While the line was in use, there was a bridge connecting Midsomer Norton to Radstock, over Silver Street seen in the plan. However, the bridge was removed after the line's closure, and elevation of the street was increased. It is in plan to construct a new bridge there at later stages but for the early stages it seems to be impossible to build a new bridge with the limited funds.

So, Midsomer Norton station is a terminal now and as a first step, it is proposed to build a siding and a depot for the terminating trains. The proposal of the siding and the shed is shown in figure 3.

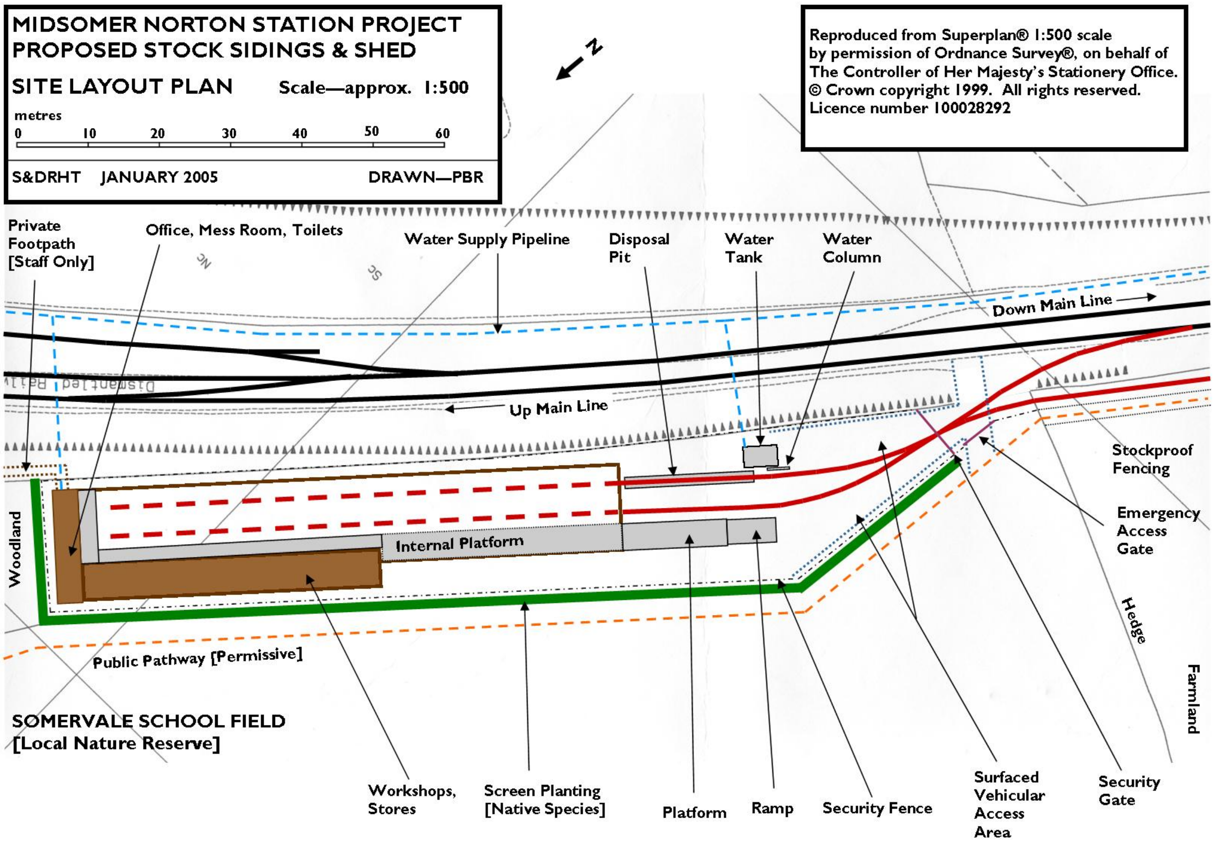


Figure 3 Proposed Siding and Shed Layout (S&DHRT)

The organisation who is involving this reopening process is the Somerset and Dorset Heritage Railway Trust which is run by volunteers. S&DHRT is a charitable trust and has around a thousand members.

S&DHRT have a very limited budget and they need volunteers to undertake the site investigation. As this study is funded by University of Birmingham Railway Systems Engineering and Integration course budget, this study's importance increases as it helps a charity as well as it is a contribution to the scientific literature.

3 Literature Review

3.1 Introduction

This chapter covers literature review of this study. In general terms, site investigation is analysed, geology of the area and the material Mercia Mudstone, which is present on site of the proposed railway track, is investigated. Several sources are searched, studies of different people are found and they are analysed by the author in this chapter.

3.2 Site Investigation

Site investigation is the process in which necessary geotechnical information is obtained and the conditions of the site is evaluated to see how it affects construction or performance of a civil engineering work such as building, railway, highway, bridge, tunnel, etc.

Although investigating the behaviour of earth and rock for engineering purposes is supposed to be started in Roman times, it is admitted that important improvements in analysis happened in the 18th century, when required large defensive revetments caused early work on retaining walls. An early work of important understanding of the behaviour of soil was represented in Coulomb's paper, which was published in 1776. Those results are still valid and in use (Heyman, 1972).

However, in the 19th century, further developments were processed. Pre-loading was used in the construction of Caledonian Canal in 1809 to squeeze out the water and consolidate the mud. Also, Stephenson used drains for the purpose of lowering pore pressures in the construction phase of the Chat Moss embankment on the Liverpool and Manchester Railway between 1826 and 1829 to consolidate the ground on which the railway track was to be built. (Clayton et al, 1995)

In the beginning of the 20th century, several failures happened. Slope failures on the Panama Canal, and landslides during a railway construction and some other occasions led to foundation of many research groups in different countries. However, Casagrande (1960) indicates the progress of modern soil mechanics started between 1921 and 1925, when Terzaghi produced considerable amount of work about pore pressures in clay during loading, and their dissipation during consolidation and also when he published his book. (Clayton et al, 1995)

It was stated by Terzaghi in 1953 that, all the knowledge, which is required for a rational analysis of the observational and experimental data, has been obtained. (Terzaghi, 1953) According to Clayton et al (1995), there has not been many changes in this field since 1950.

The last British standard about site investigation was published in 1999, which is named BS5930: "The code of practice for site investigations". It gives guidance on legal, environmental and technical issues associated with site investigation and includes a part about the description and classification of soils and rocks (BS5930, 1999).

Also, the other standard which is followed in this study is BS1377: "British Standard Methods of test for Soils for civil engineering purposes" which is also guidance for the necessary procedures on laboratory testing of soils (BS1377, 1990).

3.3 Engineering Geology

According to the data from British Geological Survey, it is found that the bedrock for the complete site of the investigation is formed of Mercia Mudstone. The photo indicating the Mercia Mudstone is the material underlying the site location is shown in figure 4. This was also confirmed by the study of Lamb (2013). It can also be understood by seeing the conditions of the material on site. The BGS documents showing the borehole recordings near the site are presented in the appendix.

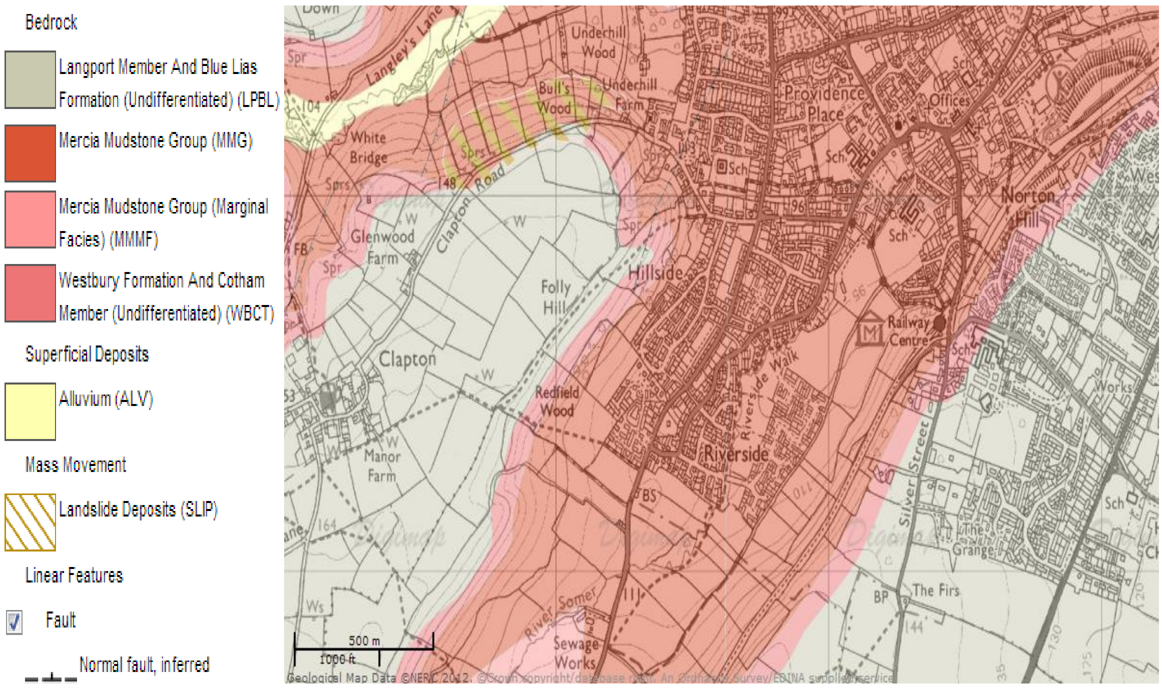


Figure 4 Engineering Geology of the site (British Geological Survey)

3.4 The Mercia Mudstone

The Mercia Mudstone Group, which is also known as Keuper Marl Series, is mostly a series of Mudrock layers existing under much of central and Southern England and on which many urban areas and their associated infrastructure are built (British Geological Survey). It is very important to the construction industry since it is commonly faced in many civil engineering works including foundations, excavations and earthworks. (Nathanial and Rosenbaum, 2000). It is the mudstone that is most frequently encountered in construction and it is the mudstone that this study is mainly concerned with. Its exact name is 'mudstone of the Mercia Mudstone

Group' or 'Mercia Mudstone Group mudstone'. However, author tends to use Mercia Mudstone as it is not as repetitious as the original term and it makes the report easier to read. (Hobbs et al, 2002)

Mercia mudstone seems to cause some critical geotechnical problems compared to other, more plasticity clays (Jones, 2000). However, it is essential for construction industry, as often occurs in engineering activities involving foundations, excavations and earthworks.

Figure 5 shows where Mercia Mudstone is found throughout the UK. The numbered areas in the figure correspond as:

1.Wessex Basin; 2.Somerset/Avon/South Wales; 3.Worcester/Knowle Basins; 4.Stafford/Needwood Basins; 5.Cheshire Basin; 6.West Lancashire; 7.Carlisle Basin; 8.East Midlands/NE England.

Figure 5 Outcrop of the Mercia Mudstone Group in Britain (Hobbs et al, 2002)

3.4.1 Classification

As reported by BGS, sampling and testing is difficult because of the fact that the material may not be fit for either soil or rock techniques. Measuring particle size grading and plasticity can be problematic as it is difficult to disaggregate the component particles sometimes.

Swelling and shrinking may be subjected by some parts of the Mercia Mudstone sequence, with changes in water content to an adequate level which leads structural damage to buildings or disruptions in some types of construction work.

Mercia Mudstone comprises sandstone beds and evaporite minerals, primarily halite (sodium chloride) and gypsum (calcium sulphate, $2H_2O$). These minerals may cause serious problems to building but they are not investigated in this study. It needs further investigation, including chemical laboratory work. (Hobbs et al, 2002)

Birch (1966) and Chandler (1969) were the early published studies on Mercia Mudstone's geotechnical features and characteristics. The former was carried out at University of Birmingham. Several laboratory and field tests were performed on samples from United Kingdom. Birch was the first to do the classification of Mercia Mudstone. It was by weathering zone. He divided it into weathered and unweathered zones. After that, Skempton and Davis worked on classification and it was used long years in most engineering applications. (Skempton and Davis, 1966) However, BS5930 (1999) offers the five-fold-approach for classification and weathering description. It suggests that classification can be defined using local experience, site specific studies or reference to established schemes. (BS5930, 1999)

The Mercia Mudstone is often defined as it has an affinity for moisture when used as an earthwork material. So, that could mean that it is susceptible to swelling and shrinkage, and long-term gradation. The moisture adsorption tests were analysed by Birch (1966). The absorbtion of water by an unsaturated mudstone might cause softening (decrease in strength) or slaking (decrease in strength and structural breakdown) or both. However, slaking usually happens because of repetition of wetting and drying cycles. No much information on swelling, shrinkage and slaking can be found in the literature.

3.4.2 Engineering Properties

Typical values for geotechnical parameters of Mercia Mudstone, studied by Chandler and Davis (1973) and Cripps and Taylor (1981) are shown in table 1. However, the methodology of the tests may affect the determination of plasticity (Hobbs et al, 2002). As noted by Birch (1966), increased remoulding using a Habert mixer to prepare samples causes higher liquid limit values.

Table 1 Outcrop of the Mercia Mudstone Group in Britain (Hobbs et al, 2002)

	Chandler and Davis (1973)		Cripps and Taylor (1981)	
Property	Unweathered	Weathered	Unweathered	Weathered
Bulk density (Mg/m3)	2.480	1.840	-	-
Porosity (%)			1-50	
Natural water content (%)	5	35	5-15	12-40
Liquid limit (%)	25	60	25-35	25-60
Plastic limit (%)	17	33		
Plastic index (%)			10-35	
Undrained shear strength Su (KPa)			130-2800	
Young's modulus, E(MPa)	250	2	100-1200	10-100
Modulus of volume compressibility m_v (m^2/MN)	0.004	0.4	0.008	
Effective friction angle, ø' (°)	>40	25	≥40	25-42

3.4.3 Index Parameters

Old and his friends studied on Mercia Mudstone in Coventry area in 1989 and found the average values of liquid and plastic limit as 35% and 20% respectively. (Old et al, 1989)

Another study on Mercia Mudstone's index parameters is carried out by Murray et al, found the liquid limit to be 34% and plastic limit to be 20%, therefore plasticity index is 14%. (Murray et al, 2001)

Changes in natural moisture content of Mercia Mudstone can be wide. Chandler (1967) found the natural moisture content to be from 22% to 25% for a Mercia Mudstone profile at Kings Norton, Birmingham. It is noted by Chandler et al (1968) that moisture content affects all physical properties such as geophysical, strength and compaction.

A study by British Geological Survey found the maximum and minimum values of particle density of Mercia Mudstone are 2.86 Mg/m^3 and 2.42 Mg/m^3, and an average of 2.69 Mg/m^3. So the particle density is accepted as 2.7 Mg/m^3 in this study. (Hobbs et al, 2002)

3.4.4 Strength

There are many studies on the strength of Mercia Mudstone researched during the mid-sixties, in the Midlands, especially at the University of Birmingham. Chandler (1967) and Chandler et al (1968) analysed the outcomes of drained and undrained triaxial tests on samples of 'Kings Norton marl' which is taken from block samples in a brick pit in Birmingham, and they researched the geological factors affecting the Mercia Mudstone's strength. Outcomes of the experiments demonstrated the effective friction angle varying from 40.5° at low stresses to 20.9° at high stresses (Hobbs et al, 2002). Also Hobbs et al (1994) processed consolidated-undrained triaxial tests on Mercia Mudstone from Gainsborough and found the values for c and ø as 14.7 kPa and 20.9°.

3.4.5 Consolidation

Typically, Mercia mudstone is known as having low compressibility and a high level of consolidation, however it is highly variable (Birch, 1966). It is noted by Birch (1966) that consolidation settlement of unweathered Mercia mudstone under normal workload is negligible. Old et al (1989) found the compressibility of Mercia mudstone, according to the consolidation of odometer data for the Coventry area, is "very low" to "medium", and the level of consolidation is "low" to "high" (Old et al., 1989).

According to Hobbs et al (2002), standard laboratory equipment for testing consolidation does not provide an adequate level of stress to obtain a complete description of the consolidation behavior, including over-consolidation ratio of hard, unweathered Mercia mudstone. (Hobbs et al, 2002)

3.4.6 Compaction

It is demonstrated by Chandler et al (1968) that there is a significant rise in California bearing ratio (CBR) with placement moisture content decrease, for example from 10% to 110% for a five percent moisture content decrease. It is easy to compact weathered Mercia Mudstone in the field, however it is difficult for hard material having low moisture content. The moisture condition value test (MCV) is commonly used as a quick indication of possible compaction behaviour. Particularly, the MM has specified MCV between 8 and 12 (Class 2A-wet) and 12-15 (Class 2B-dry) for general cohesive fill (Department of Transport, 1991). The MCV may be in relation with undrained shear strength, California bearing ratio and moisture content.

3.5 Previous Ground Investigation

A similar study to this has been done by Emeny Lamb, as a master's dissertation in March, 2013. (Lamb, 2013) This study carries her report to a further stage. She had insufficient number of samples to do all the tests wanted. Her findings for liquid limit were between 62.3% and 109%. She found plastic limit from 26.5% to 49.9% depending on the place of the sample. And the optimum moisture content of the material in the area was determined to be around 32.5% by Lamb's tests. Further tests were done in this study and compared to her findings. A suitable design is processed by the author combining Lamb's findings with his own findings. (Lamb, 2013)

4 Systems Engineering Approach

Design and development phase of any new system is seen as creative and complex process. Systems engineering is a very systematic approach that facilitates design and development of new systems. So, systems engineering can be defined as an engineering branch which involves the development of complex systems such as railways.

It is also seen as a sophisticated form of project management, unlike a holistic approach to the complex systems' creation and modification. (Schmid, 2006)

Systems engineering is also defined by International Council on Systems Engineering as "an engineering discipline whose responsibility is creating and executing an interdisciplinary process to ensure that the customer and stakeholder's needs are satisfied in a high quality, trustworthy, cost efficient and schedule complaint manner throughout a system's entire life cycle." (INCOSE, 2004). This definition explains systems engineering is needed to ensure a system works efficiently and successfully in terms of requirements, performance, management, maintenance, budget and timescale.

Schmid (2006) notes that, scope and scale of railway projects, either it is a renewal of existing structure or building of new structures, have great importance and various stakeholders are involved in railway projects over long periods of time. Due to the difficulty in obtaining funds for such projects, it is significant to serve a good management for all stages of the project. There have been many failures in high profile railway projects related to cost overruns, extended timescales, and impractical functionality after completion. The failures are usually arising from misconceptions about the nature of complex systems and absence of proper systems engineering and project management. (Schmid, 2006)

With the use of systems engineering approach in a project, design of the project can be traced at every stage, and if any failure occurs at any stage, the source of the failure can be found and it can be solved easily. This avoids extensions in time scale, over budgeting and impracticable functions.

In this study, systems engineering approach is conducted to the design of the railway sidings, analysing the stakeholders, requirements of the project, functional and physical decomposition of the requirements, and building the component at the last stage.

4.1 V – Process Model

V – process model, which is also called Vee diagram, is commonly used in systems engineering in order to represent a basis for controlling numerous different activities. It is provided as a process model in different areas including new product design, existing product development, operational maintenance and project management. (Armitage, 2007)

The diagram of Vee process in systems engineering is shown in figure 6.

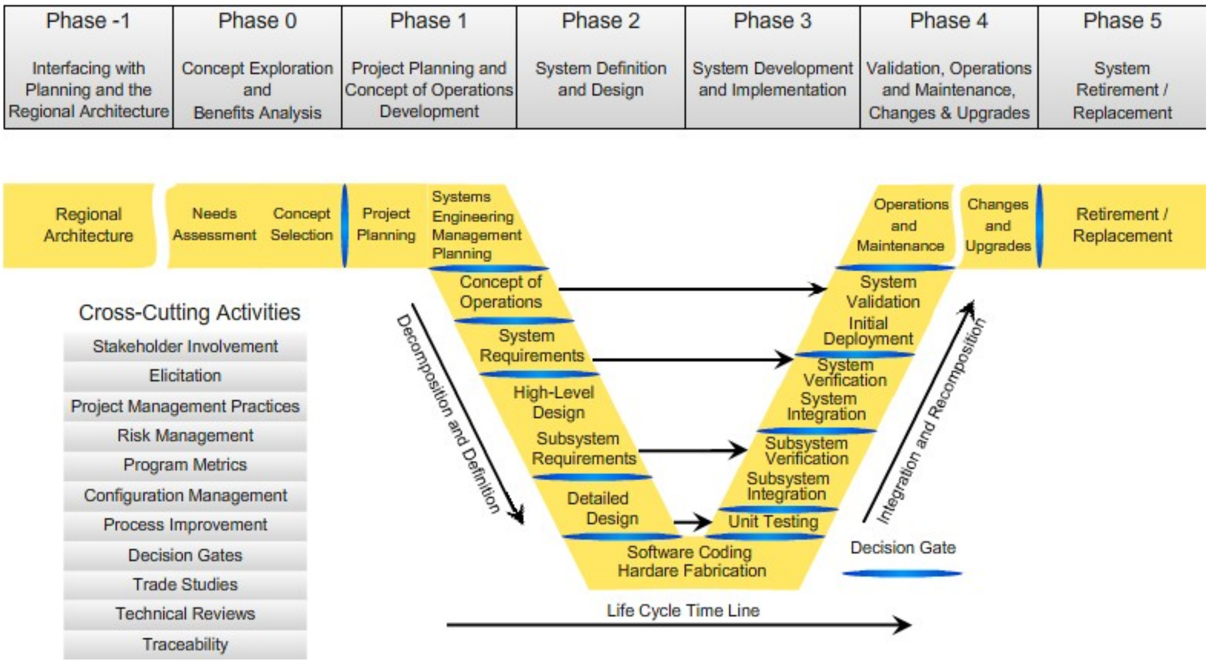


Figure 6 General Vee Process Diagram (Department of Transportation, 2009)

From the perspective of railways, the V process is to ensure reliability, availability, maintainability and safety (RAMS) of a system. If a systematic model, which is provided by V – process, is implemented properly, it ensures that a complex product can be designed to meet the market's requirements successfully. (Armitage, 2007)

There are many versions of the Vee diagram, however all provide a formal process from stakeholder analysis to operation and decommissioning. There are feedback links between the left hand side and the right hand side of the Vee diagram, which demonstrate the validation and verification stages, which are essential to the systems engineering approach and ensure traceability. A typical diagram of V - process used in systems engineering is shown in figure 6. However, author adopted a V – process with subsystems represented in figure 7.

In this approach, the main V represents the main project, which corresponds as building sidings and depot. The first phase, which is a subsystem of this main project, is ground investigation. After identifying the stakeholders and requirements of the main system;

stakeholder analysis, requirement elicitation, functional and physical decomposition and design system phases for the subsystem ground investigation were performed in this study.

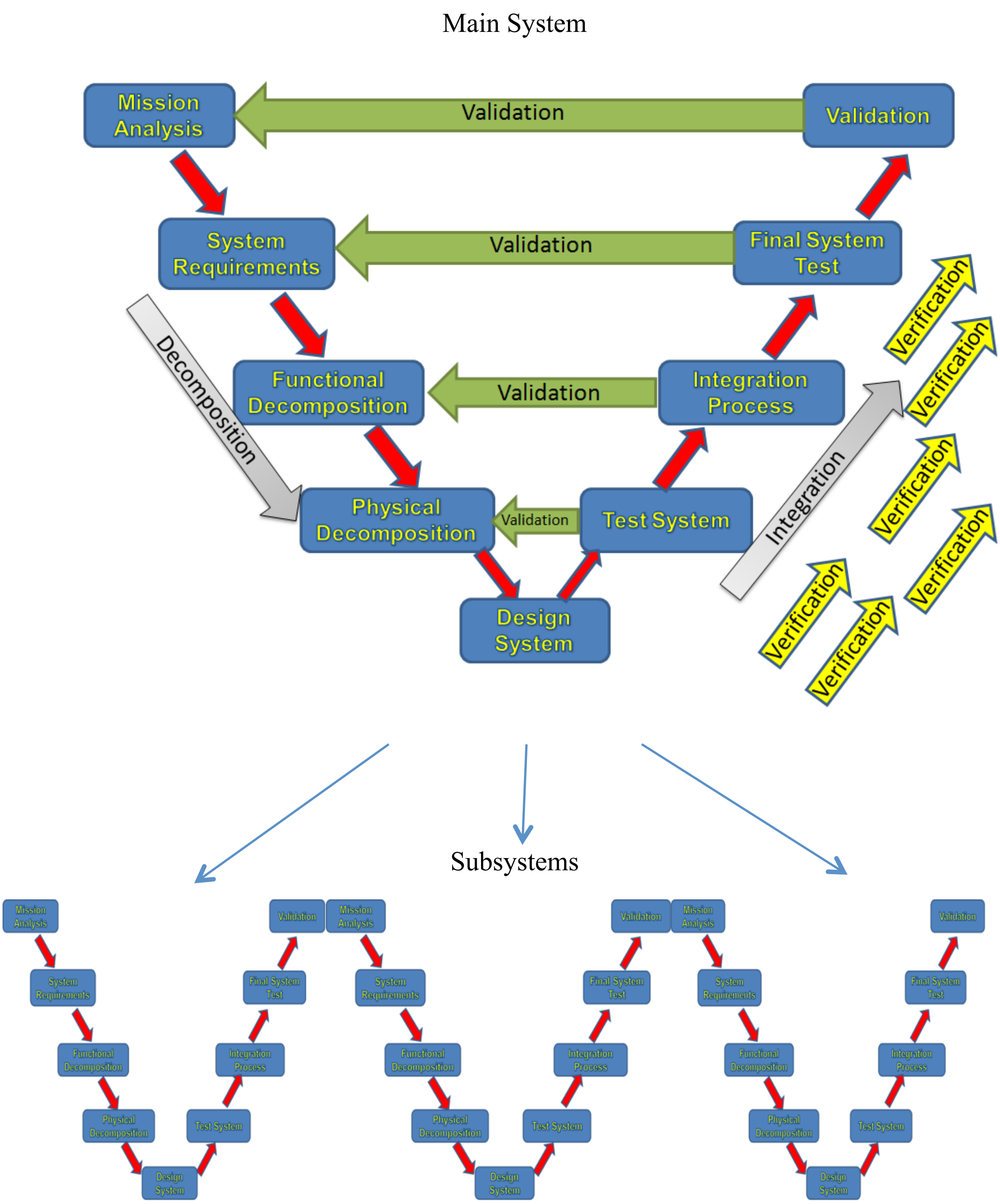


Figure 7 V - Process applied to the siding renewal project

4.2 Stakeholder Analysis

As a first step, people and institutions who are involved in the project were analysed and stakeholders of the main system were defined. Stakeholders of the main Vee are:

1. Passengers
2. Staff
3. Railway Inspectors
4. Town Council
5. English Nature & English Heritage
6. Environment Agency
7. Neighbours
8. Tourists
9. Constructor
10. Maintainer
11. Finance

After that stakeholders of the sub-Vee were identified:

1. Dissertation author: Usame Ekici
2. Academic supervisor : Dr. Gurmel Ghataora
3. Industrial Advisor: John Dora and Peter Russell
4. Surveyor: British Geological Survey
5. Archaeologist
6. Excavator: JCB owner and operator
7. Finance : University of Birmingham

Author has undertaken a ground investigation. Engineering excellence and experience is necessary to build this project. Supervision was performed by Dr. Gurmel Ghataora of University of Birmingham, and plans and specifications were obtained from John Dora and Peter Russell of S&DHRT. Information on geology of the area was obtained from British Geological Survey's borehole recordings and other surveys. However there is also a need for an archeological survey, which has always been under consideration but due to the limited funds, could not be performed yet. An excavator to dig trial pits is needed, the excavator machine and the operator are from S&DHRT. To perform the ground investigation project, expenses are covered by University of Birmingham Railway Systems Engineering and Integration Department.

4.3 Requirement Elicitation

Requirements are the foundation of systems. They determine what the system must do and help development of the system progress. Requirements are used to check if the system is built correctly. The requirements development phase identifies the activities necessary to produce a set of complete and verifiable requirements. (Department of Transportation, 2009)

Requirement elicitation is an often poorly completed process of systems analysis. Mistakes which were made in elicitation usually result in systems failure or abandonment and this has a very large cost either in the complete loss or the expense of fixing mistakes. (Davey and Cope, 2008)

Requirements of the main project were described and tabulated with the stakeholders involved, represented in table 2.

Table 2 Requirements for the main system

Requirement	Stakeholders Involved
R1. A siding is needed to allow terminating trains to go to depot	S1, S2, S4, S7, S9, S10
R2. A depot is needed to accommodate the needed number of trains.	S1, S2, S4, S7, S9, S10
R3. The depot and track must be able to resist certain seismic forces and ground movements	S1, S2, S3, S4, S7, S9, S10
R4. Drainage system must be capable to allow the ground water to go out.	S1, S2, S3, S4, S6, S7, S9, S10
R5. Standards and rules must be obeyed	S1, S2, S3, S4, S9, S10
R6. Natural environment must be protected	S5, S6, S7, S8, S9
R7. Cost of the project must be in determined budget	S9, S10, S11

After that, requirements for the subsystem ground investigation were determined.

Table 3 Requirements for the sub system

Requirement	Stakeholder Involved
R1. A good scientific investigation must be processed	S1, S2, S3
R2. Pits need to be dug to observe the ground conditions	S1, S2, S6
R3. The ground parameters must be obtained by doing laboratory tests and	S1, S2
R4. The ground characteristics must be analysed to know whether the ground is suitable for building the track and the depot	S1, S2, S3
R5. Consultancy is needed from experienced people	S2, S3
R6. A good map and plan of the project area must be obtained	S3, S4
R7. An archaeological survey is needed to know if there are any valuables underneath the ground	S5
R8. The cost of the project must be in determined budget	S7

4.4 Functional Decomposition

In this stage, functional design of the product is commenced in response to the system requirements. Noted by Roberts (2005), functional decomposition helps the system requirements to be developed in the form of a hierarchy of functional requirements which can introduce the information flow between functions. However, it does not directly tell how to implement the derived subsystems physically.

In order to ensure that all requirements are covered by functions and sub functions, a cross-reference check between the system requirements and the functional decomposition is processed in the concluding stage of the functional decomposition. It is also important to ensure traceability of the system design. (Armitage, 2007)

Top function is determined to be “accommodate trains safely”. And the sub functions are “minimize earthwork” and “stabilize ground”. There are also sub functions of these sub functions which are called sub-sub functions. All the functions with the related requirements are demonstrated below.

- Top function: Accommodate Trains Safely
    - Sub-function 1: Stabilise Ground
        - Sub-sub function 1.1: Drain water appropriately (R2, R3, R4)
        - Sub-sub function 1.2: Guide the track (R3, R4)
        - Sub-sub function 1.3: Avoid flood, landslide (R4, R5)
        - Sub-sub function 1.4: Discover soil (R2, R3, R4, R5)
        - Sub-sub function 1.5: Form a safe cut / fill slope (R1, R5, R6)
    - Sub-function 1: Minimise Earthwork
        - Sub-sub function 2.1: Allow a suitable location for sidings and depot (R2, R6)
        - Sub-sub function 2.2: Form a proper gradient (R5, R6)
        - Sub-sub function 2.3: Form a suitable cut / fill slope (R1, R5, R6)
        - Sub-sub function 2.4: Balance the amount of cut and fill (R1, R6)
        - Sub-sub function 2.5: Minimise the cost of excavation and hauling (R1, R6, R8)

4.5 Physical Decomposition

In this stage, physical elements, which will be needed to implement the necessary functions, are analysed so that it helps the development of the product’s functional definition. Each function or sub function is allocated to a specific physical sub system or assembly, in the functional decomposition stage. (Shisko and Chamberlin, 1995)

After functional decomposition, physical decomposition was processed. Physical factors of the system were analysed according to the functions defined in the functional decomposition. So, all the physical parts are related to the functions analysed above.

Table 4 Physical Decomposition

Physical Component	Function
Ballast	1.1, 1.2, 1.3
Rail	2.1, 2.2
Excavator	1.4, 1.5
Drainage pipes	1.1, 1.3
Roller	1.5, 2.2
Laboratory Equipment	1.4
Computer software	1.5, 2.2, 2.4

4.6 Design System

In the design stage of the systems engineering V – process, the physical components are evaluated and the engineering design is performed. It is the last stage on the left hand side of the Vee diagram, and the last stage of the systems engineering approach of this study. A further study can be processed after the construction phase completed, to complete the Vee diagram's right leg.

Design phase is performed and demonstrated in chapter 7.

5 Subsurface Exploration and Sampling

Subsurface exploration and sampling is the first phase of the in-field part of ground investigation, after desk study. A walkover survey is included in this process. The first walkover survey was performed on 27th April 2013 by the author.

As a first step, the site was discovered walking around. Peter Russell and John Dora of S&DHRT were present on the site, they gave a briefing, and the author obtained necessary information about the geology and use of the site. Also, the museum and the shop in the area were seen and some other information was gathered talking to volunteer workers, seeing the objects and reading the information beside the pictures on the walls in the museum. Also, a short trip with one of the old trains in the area was taken to discover the railway track and further the site. A whole day was spent in the site for the first walkover survey with walking around, taking pictures and collecting information. The picture taken by author and presented in figure 8 shows footpath and location of the proposed siding. The white posts show boundaries of the siding.

It is also seen that there are some trees which might need to be cut down. However, if they have to be moved, the nesting season of the birds should be followed and they should be cut down in the right season, to protect the ecological environment.

Figure 8 Footpath and the proposed location of siding

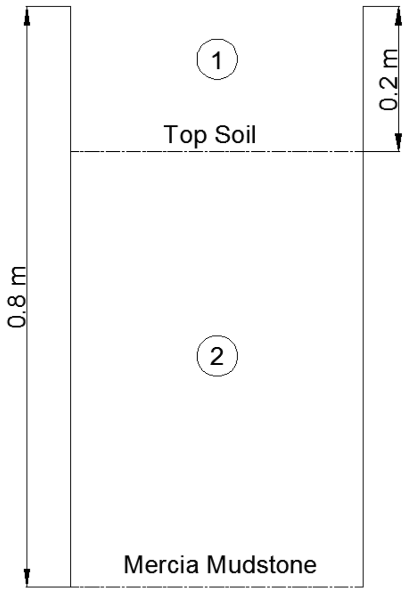
The second walkover survey was conducted on 23rd July 2013 with Dr. Gurmel Ghataora and Peter Russell the day before the sampling. This time, a shorter walkabout was processed and locations of the trial pits were determined. The excavator of S&DHRT was used by a volunteer operator from S&DHRT to dig trial pits. As it was a small capacity excavator and the ground was going stiffer by depth, the deepest samples were from 0.8 m below the ground surface. Also, undisturbed samples could not be obtained since the ground was so stiff that undisturbed sample tubes could not be hammered.

8 bags from 4 pits were filled with samples. Each bag was numbered and labelled with the reference number of trial pit and the depth of top and bottom of the sample. The bags were tied carefully to prevent any air coming in. It was tried to get as many samples as possible in order to have enough number of specimens for necessary laboratory tests. Because of limited number of available equipment, a detailed in-situ testing could not be processed.

To ensure a safe work in the site, a risk assessment was prepared by the author before going to the site for sampling.

Figure 9 Excavating Trial Pits

When the pits were excavated, different layers of the soil were seen. Height of the layers were measured and are represented in the figures below, drawn by the author. Table 5 shows the depth of the samples with the number of trial pits and bags. Also, a photo showing the layer heights is represented in figure 14.



Layer 1 shows topsoil and layer 2 represents very stiff reddish brown, grey/green mottled silty clay.

Excavating stopped at 0.8 metres due to difficulty in excavating deeper.

Bag 1 was taken from 0.55 m depth and bag 2 was taken from 0.75 m.

Figure 10 Cross-section of trial pit 1

Layer 1 is topsoil and layer 2 shows stiff fissured greyish brown silty clay with some medium to coarse platy limestone.

Layer 3 represents very stiff reddish brown silty clay with some grey green colourants.

Bag 1 was taken from depth of 0.25 m, bag 2 was taken from depth between 0.55 and 0.65 m and bag 3 was taken from 0.65 – 0.75 m.

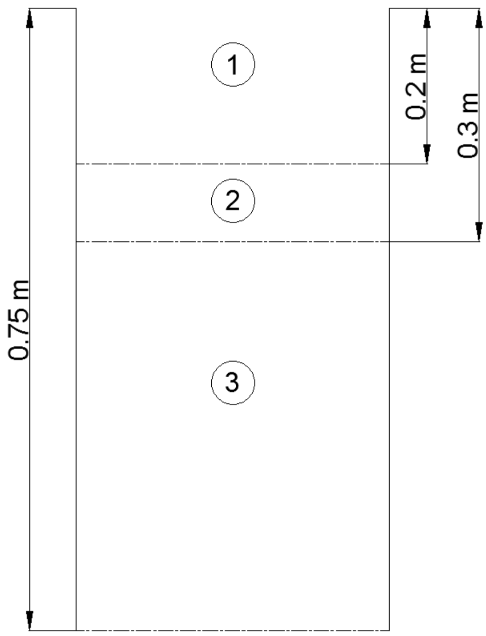


Figure 11 Cross-section of trial pit 2

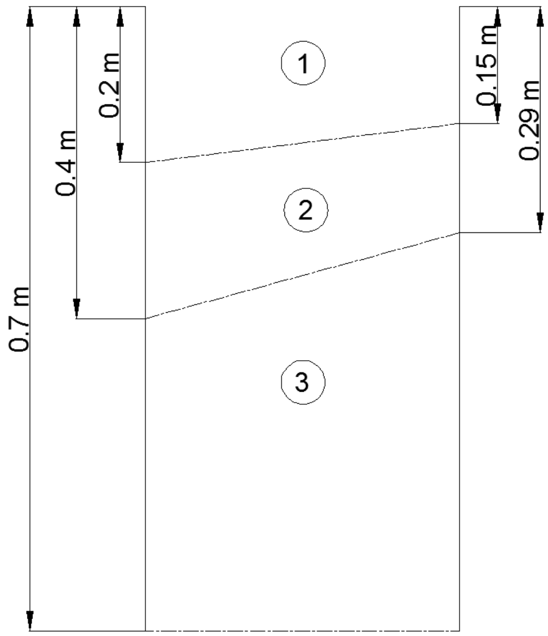


Figure 12 Cross-section of trial pit 3

Layer 1 is topsoil.

Layer 2 shows stiff pale brown silty clay with some medium to coarse platey limestone.

Layer 3 represents very stiff reddish brown, with some grey/green mottled silty clay – Mercia Mudstone.

Bag 1 was taken from 0.6 m depth.

Layer 1 is topsoil.

Layer 2 represents blocky very stiff reddish brown mudstone with some grey/green colourants with depth becoming very stiff silty clay.

Bag 1 was taken from 0.5 m and bag 2 was taken from 0.8 m.

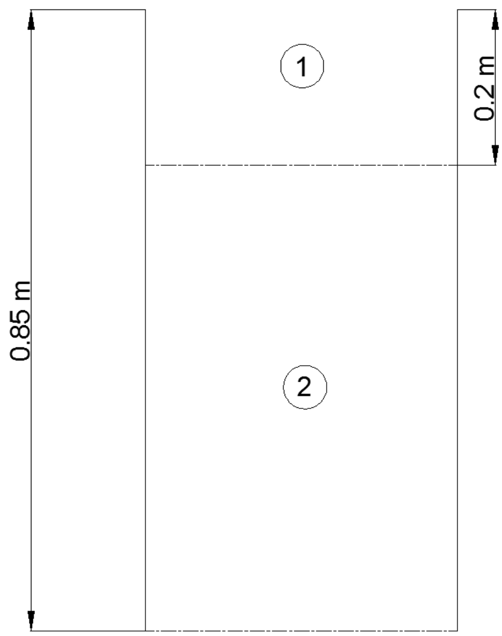


Figure 13 Cross-section of trail pit 4

Table 5 Depth of the bag samples

		Trial Pit			
		1	**2**	**3**	**4**
Bag	**1**	0.55 m	0.25 m	0.6 m	0.5 m
	2	0.75 m	0.55 - 0.65 m		0.8 m
	3		0.65 - 0.75 m		

Figure 14 Measuring the soil layers

As the material from all 4 trial pits was same, it was not seen necessary to dig more trial pits towards the area where the shed will be. So, when those pits were excavated, they were all filled back and their location was obtained by using a tape measure and referencing the posts showing boundaries of the proposed siding. It is drawn in Autocad by the author, using the level survey map which was undertaken by a retired trained surveyor of S&DHRT. The scale is not marked in the map. However, the scale was obtained by the author measuring the track gauge in the map. The track gauge is drawn 2.85 mm, and its real length is 1435 mm. So, if 1435 is divided by 2.85, it is equal to 503.5. Considering the gauge cannot be exactly measured, the scale is determined to be 1:500. Layout of the trial pits is demonstrated in figure 15.

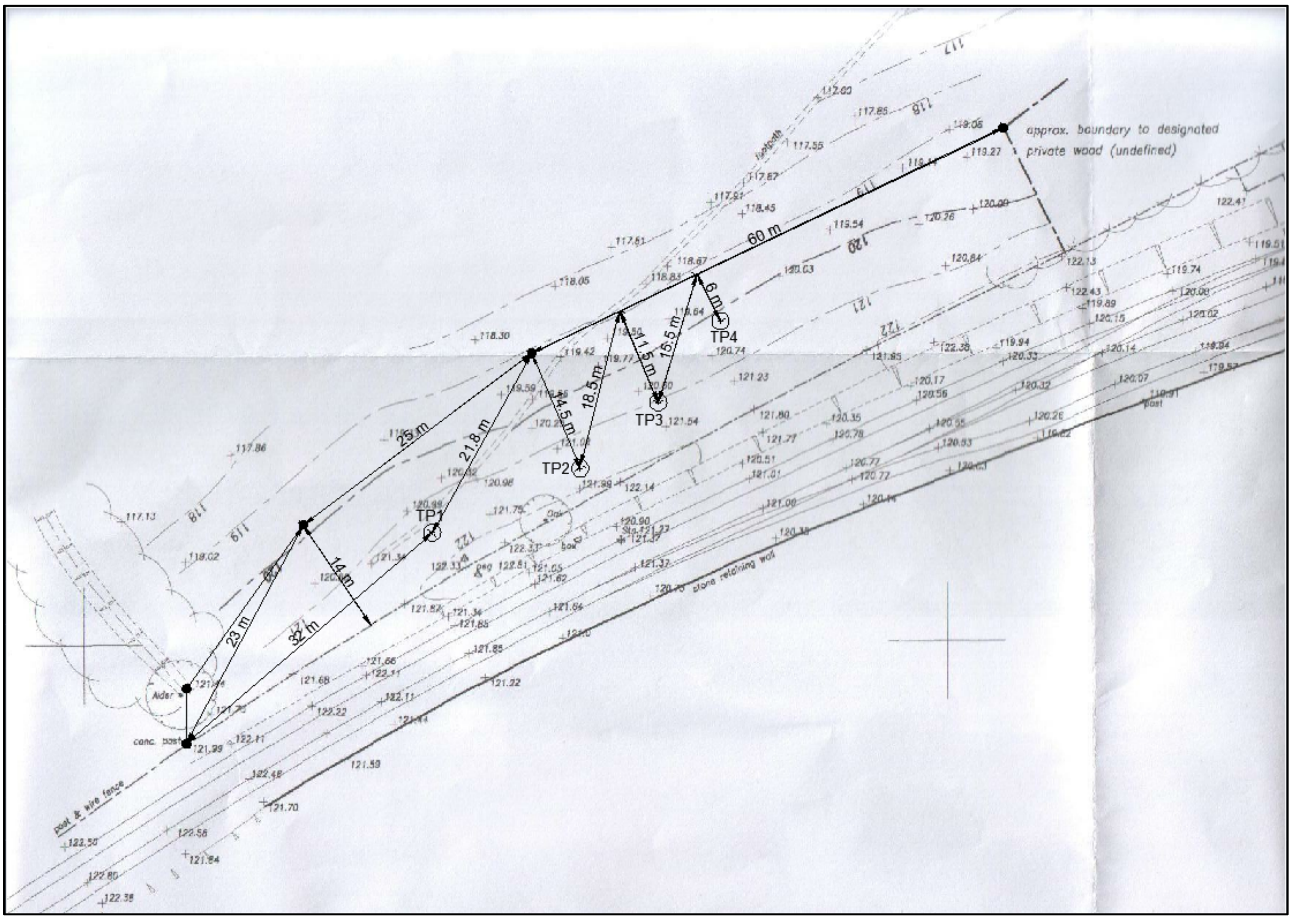


Figure 15 Layout of trial pits

6 Laboratory Testing

6.1 Introduction

After taking samples from the site, they were brought to the civil engineering laboratory of the University of Birmingham. Laboratory testing started with moisture content tests on 24th July 2013. The schedule with the standards followed is shown in table. 6 Around a month of full time work was processed in the laboratory.

Before the tests, several sources were searched and read to know about the test procedures. Also, several videos showing test procedures were seen on the internet to get familiarity with the test equipment and test process. Training was done before each test to ensure the correct use of the equipment and correct procedure of the tests.

Table 6 Laboratory Testing Schedule

Time	Test	Protocols and Methods	property measured
24/07/2013 - 02/08/2013	Index tests	BS 1377: Part 2: (1990) British Standard Methods of test for soils for civil engineering purposes, Part 2: Classification tests, British Standards Institution, London.	Natural Moisture Content, Liquid Limit, Plastic Limit
02/08/2013 - 07/08/2013	Proctor test	BS 1377: Part 4: (1990) British Standard Methods of test for soils for civil engineering purposes, Part 4: Compaction-related tests, British Standards Institution, London.	Maximum dry density, optimum moisture content, compaction
02/08/2013 - 07/08/2013	Shear vane test	BS 1377: Part 8: (1990) British Standard Methods of test for soils for civil engineering purposes, Part 7: Shear strength tests (total stress), British Standards Institution, London.	Shear Strength
12/08/2013 - 19/08/2013	Triaxial test	BS 1377: Part 8: (1990) British Standard Methods of test for soils for civil engineering purposes, Part 8: Shear strength tests (effective stress), British Standards Institution, London.	Undrained shear strength

6.2 Natural Moisture Content Test

6.2.1 Scope of the Test

As the first step of laboratory testing, the moisture content (or water content) of the each sample was measured following natural moisture content test procedures. This test is the most basic test for soil samples, as it provides a very useful method of classifying cohesive soils and analysing their engineering properties. However, moisture content by itself is not enough to define the state of consistency of a clay soil. Liquid and plastic limits are related to the moisture content to do this definition (Head, 1980).

There is water in nearly all type of naturally occurring soils as water is a part of the soil structure. The moisture content of a soil corresponds as the amount of water within the pore space between the soil particles, and it is assumed to be removable by oven drying at 105-110°C. Soil sample becomes 'dry soil' when it reaches to constant mass, waiting in an oven for about 12-24 hours (Head, 1980).

6.2.2 Procedure

After leaving soil samples in the oven to dry for 24 hours and reaching constant mass, the difference in weight between the wet sample and dry sample is noted. This amount is equal to the weight of the water, which is contained in the soil in its natural environment. Water content is simply calculated as the ratio of 'mass of water removed by drying' to the 'mass of dry sample', and it is usually stated as a percentage.

6.2.3 Calculations

If the mass of water is shown by m_w, and mass of dry sample is m_d, then water (moisture) content is expressed as:

$$\omega = 100 \text{ x } (m_w/m_d)$$

All 8 samples were labelled properly and put into oven after the weight of trays and samples were measured. They were taken and weighed again in the next day. The change in weight is calculated and water content of each sample is found. Moisture content values for all the samples are shown in table 7. More detailed results of the tests are shown in the appendix.

Table 7 Moisture Content Values

Sample No	Water Content (%)
TP1B1 (0.55 m)	27.98
TP1B2 (0.75 m)	36.14
TP2B1 (0.25 m)	30.38
TP2B2 (0.55-0.65 m)	33.37
TP2B3 (0.65-0.75 m)	38.19
TP3B1 (0.6 m)	34.94
TP4B1 (0.5 m)	38.20
TP4B2 (0.8 m)	40.57

6.3 Atterberg Limits

After identifying moisture contents, each sample was subjected to Atterberg Limits tests.

6.3.1 Scope of the Tests

Liquid limit, plastic limit and shrinkage limit are known as Atterberg limits, since the Swedish scientitst Dr. A. Atterberg described them for the classification of soils in 1911. They were done by simple hand tests in the early times. However, the procedures of the tests were improved and modernised in 1932 by professor A Casagrande, and still the same test procedures are used worldwide (Head, 1980).

As noted by Head (1980), liquid and plastic limits serve the most suitable way of identification and classification of the fine-grained cohesive soils. Because clay particles are so small that they cannot be examined visually and particle size analysis tests cannot say anything about the type of the clay although they provide quantitative data on the particle size ranges and the amount of clay present.

The atterberg limits can be used to see the relationship between the layers of soil, which occur in different areas of a site, or to analyse how soil properties change in a limited zone. According to the outcomes of limits tests, the selection of soils for use as compacted fill can be decided in different kinds of earthwork construction. (Head, 1980)

High plasticity clays are usually more compressible, and consolidate over a longer period of time under load than low plasticity clays. Compacting clays of high plasticity is also more difficult than clays of low plasticity, when used as fill material. Atterberg limits and natural moisture content tests usually give enough indication of the nature of clay soil for many simple applications if the geological history of the soil is also known. If more information is needed by further testing, the results of the limits tests are used as a first stage to decide on the selection of samples for further tests. (Head, 1980)

6.3.2 Procedure of Liquid Limit Test

First of the Atterberg limits tests is the liquid limit test. There are two ways of doing the liquid limit test. First one is called Casagrande method which uses the Casagrande apparatus, and the second is cone penetrometer test. Usually the results of the both tests are nearly same for liquid limit values of up to 100%. However, above 100%, the Casagrande method gives slightly higher values than cone penetration. Cone penetrometer method has been used in Britain more commonly than the other. Because it is believed to be more precise as the probability of laboratory errors is less with this method.

The cone penetrometer method is based on measurement of penetration of a standardised cone, which has a specified mass. The liquid limit value is simply the moisture content of sample at which the cone penetrates 20 mm into the soil sample. At least four tests are carried out for one sample with different moisture contents to plot a graph with the values of penetration in millimetre on the x-axis and the moisture content as percentage on the y-axis. Then, a line is drawn which is best fit to those points, and the value of moisture content which intersects with the line at 20 mm penetration is determined as liquid limit.

Cone penetrometer method was followed in this study. Before beginning the test, all the samples were oven dried and subjected to crushing. After crushed, samples were sieved through 425μm sieve, mixed with suitable amount of water and tests started. A photo is presented in figure 16 showing the apparatus used in the liquid limit test. Additional care was taken not to leave any air gaps in the soil sample, and to make a smooth surface after the cup was filled, in order to obtain correct and precise results.

Figure 16 Liquid Limit Test Apparatus

For the first test, it was seen that the first reading of the penetrometer was below 15. For the other samples, some more water was added and when it was seen to be below 15, it wasn't taken into account and a new test was processed to obtain readings between 15 and 25 instead of that test.

6.3.3 Calculation of Liquid Limit Test Results

To obtain liquid limit, results of each penetration are tabulated and average value of penetration in millimetres for each repeat is plotted in a graph. A straight line which is best fit for all 4 results is drawn and the equation of the line is drawn. From the equation, the value which corresponds to 20 mm cone penetration is obtained as liquid limit. A table and a graph of liquid limit test are presented below as an example, and all results are presented in the appendix to ensure clarity of the dissertation.

Table 8 A typical table of liquid limit test results obtained from sample TP2B2

TP2 B2				
LIQUID LIMIT Test No	1	2	3	4
Dial gauge reading	15.9 15.7	18.9 19.8 19.4	22.3 22.8	23.4 23.8
Average Penetration	15.8	19.4	22.55	23.60
Container No	UA1	UA2	UA3	UA4
Mass of wet soil + container (g)	12.99	13.00	14.09	14.30
Mass of dry soil + container (g)	8.76	9.15	9.10	9.23
Mass of container (g)	3.23	4.43	3.21	3.39
Mass of moisture (g)	4.23	3.85	4.99	5.07
Mass of dry soil (g)	5.53	4.72	5.89	5.84
Moisture content (%)	76.49	81.57	84.72	86.82
Liquid Limit	81.98			

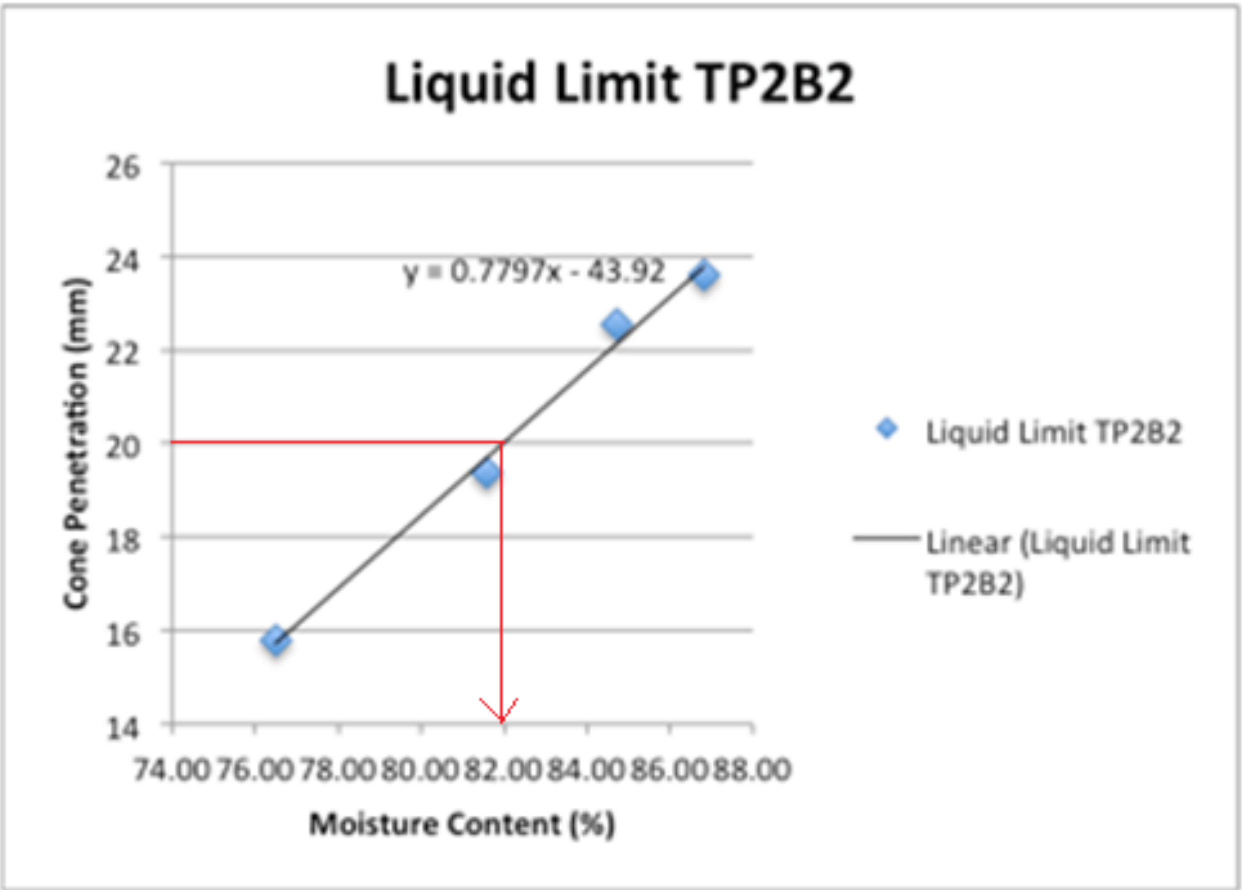


Figure 17 A typical graph of liquid limit test results obtained from sample TP2B2

Overall results of liquid limit test are shown in table 9.

Table 9 Liquid Limit Values

Sample No	Liquid Limit (%)
TP1B1 (0.55 m)	90.13
TP1B2 (0.75 m)	88.94
TP2B1 (0.25 m)	63.66
TP2B2 (0.55-0.65 m)	81.98
TP2B3 (0.65-0.75 m)	87.82
TP3B1 (0.6 m)	82.48
TP4B1 (0.5 m)	71.21
TP4B2 (0.8 m)	83.92

6.3.4 Procedure of Plastic Limit Test

Plastic limit is defined as the lowest moisture content at which soil is plastic. It is determined by standard plastic limit test. Plastic limit is expressed as percentage and is equal to the moisture content at which the soil sample can form a 3 mm cylinder without cracks. Consistency of the soil, which is about to be in transition from plastic form to loose-solid form, is determined by plastic limit.

According to British Standards (BS 1377, 1990) the plastic limit is defined as the empirically found moisture content at which a soil is too dry to be plastic. It presents a cohesive soil classification when it is used in conjunction with the liquid limit to determine the plasticity index. The values obtained are plotted against the liquid limit on the plasticity chart to give an understanding of the soil's classification. It is also stated in BS 1377 (1990) that there might be some variability in results since the results are dependent on the judgement of the operator.

6.3.5 Calculation of Plastic Limit Test Results

Plastic limit tests were done on the crushed and water-mixed samples. 3 of 20 gram samples were taken and rolled in the palm first, and then on the glass. It gets certain moisture content when it can form a 3 mm cylindrical shape, without cracks. When the samples became 3 mm cylinders, they were taken to trays and the weights are measured and noted. 4 different trays were used to measure the moisture content in order to be precise. They were all put into oven to fully dry to be able to measure the moisture content. After measuring moisture content of the each tray, the average value was calculated and determined to be plastic limit. A photo of samples ready for oven drying to determine LL and PL is represented in figure 18.

Figure 18 LL and PL samples

Results of the plastic limit test are shown in table 10. Again an example of plastic limit test results and overall results are shown below and all detailed results are represented in the appendix.

Table 10 A typical table of plastic limit test results obtained from sample TP2B3

PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	
Mass of wet soil + container (g)	15.60	14.79	15.01	13.19	
Mass of dry soil + container (g)	12.36	11.79	11.88	10.50	Sample No: TP2B3
Mass of container (g)	5.52	5.39	5.51	4.80	
Mass of moisture (g)	3.24	3.00	3.13	2.69	
Mass of dry soil (g)	6.84	6.40	6.37	5.70	
Moisture content (%)	47.37	46.88	49.14	47.19	47.64

Table 11 Plastic Limit Values

Sample No	Plastic Limit (%)
TP1B1 (0.55 m)	50.71
TP1B2 (0.75 m)	47.34
TP2B1 (0.25 m)	37.92
TP2B2 (0.55-0.65 m)	48.94
TP2B3 (0.65-0.75 m)	47.64
TP3B1 (0.6 m)	43.96
TP4B1 (0.5 m)	42.40
TP4B2 (0.8 m)	54.62

6.3.6 Plasticity Index

As mentioned above, classification is produced by means of the plasticity chart. Liquid limit is plotted as ordinate against plasticity index as abscissa. The A Line, which is the standard line in plasticity chart, is drawn by the equation below:

$$PI = 0.73\ (LL-20)$$

This line was achieved by experimental studies and does not give a very clear indication of boundary between soil types; however it represents a useful reference datum.

Clays are analysed in five zones by British Standards (BS5930, 1999). Those zones are:

The clays having less than 35 liquid limit are: Low plasticity clays (CL)

The clays having liquid limit between 35 and 50 are: Medium plasticity clays (CI)

The clays having liquid limit between 50 and 70 are: High plasticity clays (CH)

The clays having liquid limit between 70 and 90 are: Very high plasticity clays (CV)

The clays having more than 90 liquid limit are: Extremely high plasticity clays (CE)

As seen in figure 19 the material is found to be between high plasticity and very high plasticity.

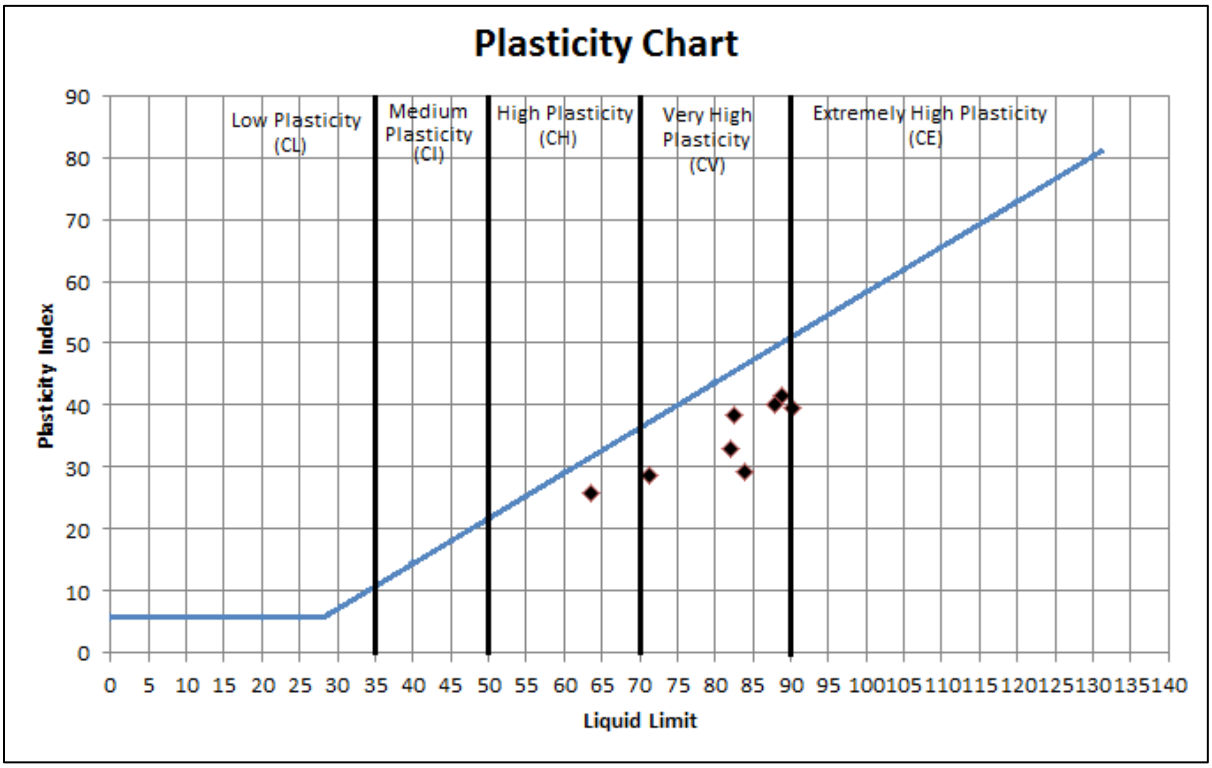


Figure 19 Liquid Limit-Plasticity Index Relationship of Mercia Mudstone from the site

6.4 Standard Compaction Test

6.4.1 Scope of the Test

Compaction is defined by Markwick (1944), as the process which leads the solid soil particles to be packed more closely together mechanically, and increasing the dry density. (Markwick, 1944).

The association between moisture content and dry density of a soil on which, a given compaction effort is applied, is obtained from laboratory compaction tests, and it gives reference datum properties and control of soil placed as fill. (Williams, 1949)

Soils are needed to use as fill material in many civil engineering projects. Compacting soil to a dense state is commonly required when it is used as an engineering fill, because it is necessary to have satisfactory engineering properties which would not be gained with loosely placed material. The data derived from compaction tests are as follows. First is the relation between dry density and moisture content of a soil, for a certain degree of compactive effort. Second is the moisture content for the most efficient compaction that is at which the max dry density is obtained under that compactive effort. And third, the value of maximum dry density. The first is expressed as a graph and the latter are derived from that graph. (Head, 1980)

6.4.2 Procedure of the Test

There are three types of compaction test described by BS 1377: Part 4, each of which has procedural variations associated with the nature of the soil. The first is the light manual compaction test, which is also known as ordinary compaction test or standard Proctor test, and in which a 2.5 kg rammer is used. The second is the heavy manual compaction test which is similar but serves a much greater degree of compaction by the use of a 4.5 kg rammer with a greater drop on thinner layers of soil. (BS 1377: Part4, 1990)

The addition of air voids lines is also required for a complete compaction curve. An air voids line is a curved line which demonstrates the relationship between dry density and moisture content for soil which has a constant percentage of air voids. In this study, the air voids lines are drawn by assuming the particle density as 2.7 Mg/m^3 according to data of a previous study by Hobbs et al (2002).

The light manual compaction test is processed in this study, on 4 samples from trial pits 1, 2 and 3. The deepest samples of these 3 trial pits were selected for the compaction test. The reason why these pits were selected is that the area where trial pits 1, 2 and 3 were excavated will probably be the cut area, therefore will be compacted as will be used as fill material and the trial pit 4 will be fill area. A drawing of layout of the trial pits is shown in figure 20, as a cross-section, but all the pits are shown in the same figure.

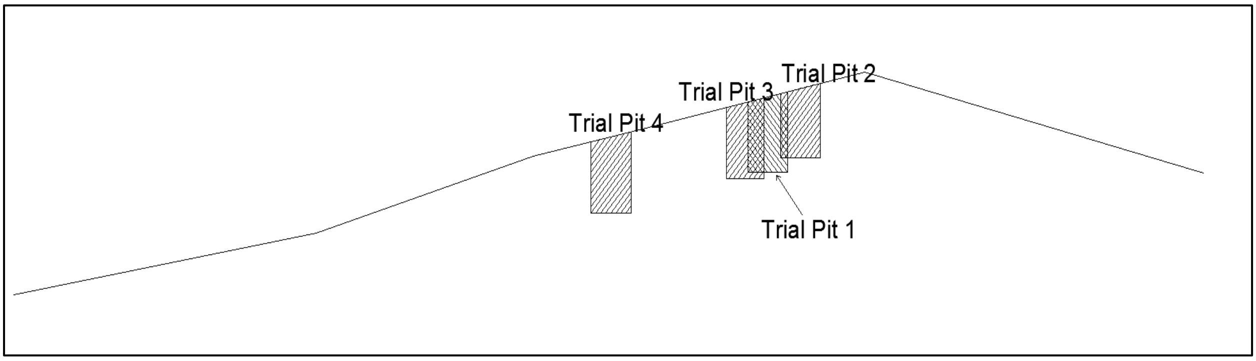


Figure 20 Layout of trial pits in cross-section

The cross-section was taken at the location of trial pit 2. Trial pit 1 is 19 metre in front of the trial pit 2, trial pit 3 is 12 metre and trial pit 4 is 23 metre behind the trial pit 2, so same as the cross-section shown in figure 20.

The samples were first taken to air dry. Then, suitable amount of water was added and mixed properly. Before starting compaction test, search had been done on the optimum moisture content value of mercia mudstone and it was found to be around 35 % by previous studies. Also, it is noted by Head (1980) that the optimum moisture content of cohesive soils is around 8-10 percent below their plastic limit.

So an approximate value of the optimum moisture content had already been known and it was tried to get as closer as to this value, and the tests were carried out with adding suitable amount of water to increase the moisture content around 4% each time. The first two samples of the compaction test (TP1B2 and TP3B1) were started with adding no water to ensure the optimum moisture content will be obtained in the 3rd or 4th trial. Then it was seen that the second sample needed 6 times of repeating to catch the maximum dry density, so the other 2 samples were started with adding 200 ml water.

6.4.3 Calculations

A 1 litre mould was used. First, dimensions (diameter, height) of the mould were measured. The diameter is noted to be 104.9 mm (see fig. 21) and the height is 115 mm, which give a 993.89 cm^3 of volume. It was weighed and oiled before the use, after that the soil sample was compacted in it in 3 layers. 27 blows with 2.5 kg hammer releasing from 300 mm height was given to each layer to compact. Once 3 layers were compacted, the extension collar was removed carefully and the excess soil was cut away, the top of the mould was levelled off. Then, it was weighed again to see the mass of soil compacted, in order to calculate the density. After that, the soil was jacked out and samples were taken from each layer, to measure the moisture content. Then, the soil was broken up and remixed with suitable amount of water to increase the moisture content by 4% for the next test. The test was repeated at least 5 times, watching the mass of compacted soil in the mould is decreased.

Figure 21 Measuring diameter of the mould

The soil samples were taken back from the oven next day, moisture content was measured. Average of the three layers' moisture content was noted, the graph was plotted using the moisture content and dry density values. Air void gaps were integrated into the graphs. Maximum dry density and optimum moisture content values are represented in table 12. Detailed results and graph of only one test are shown below, and the rest are in the appendix, to ensure the clarity of the dissertation.

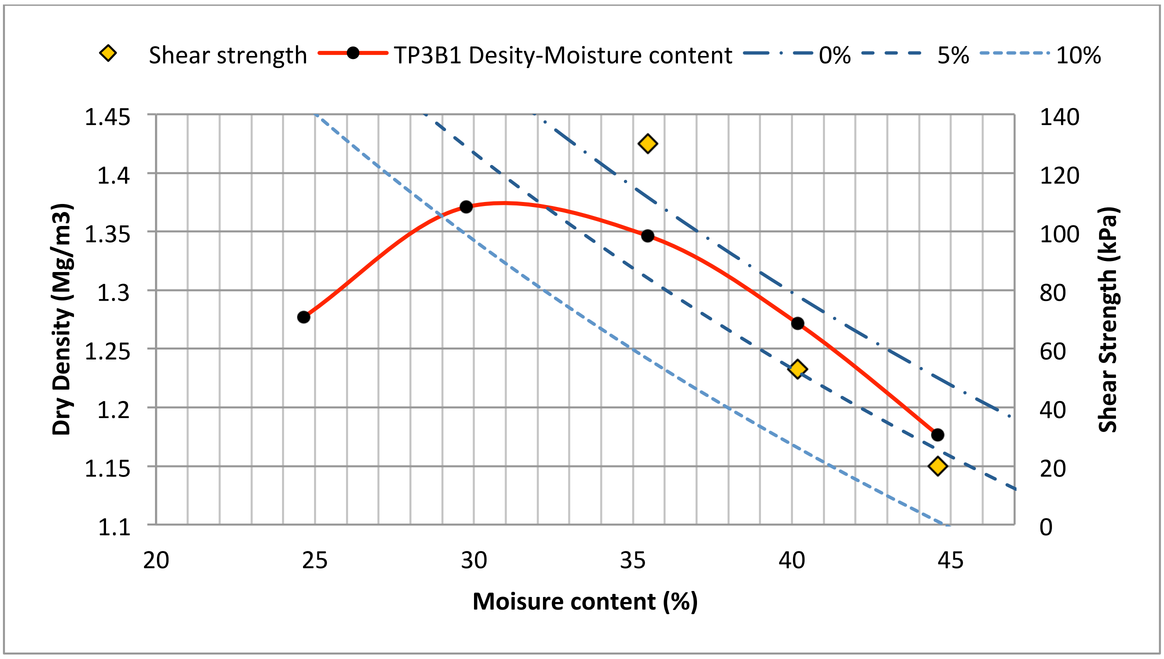


Figure 22 Typical Dry density and moisture content relationship including shear strength of Mercia Mudstone from TP3B1

Table 12 Optimum Moisture Content and Maximum Dry Density Values For All Samples

Sample No	Optimum M.C. (%)	Max Dry Density (g/cm^3)
TP1B2 (0.75 m)	37.4	1.25
TP2B2 (0.55-0.65 m)	32.8	1.34
TP2B3 (0.65-0.75 m)	32.8	1.38
TP3B1 (0.6 m)	31.2	1.37

6.5 Vane Test

6.5.1 Scope of the Test

As the ground in its natural condition was so stiff that shear vane device could not be penetrated, a laboratory shear vane test was processed. Basically, in vane test, a relative rotational movement is generated between a cylindrical volume of soil and the surrounding material. A four-blade cruciform vane is pushed into the soil and rotated. The torque which is needed to cause rotation of the cylinder of soil enclosing the vane is measured and that enables to calculate the undrained shear strength of the clay. (Head, 1980)

6.5.2 Procedure of the Test

Laboratory shear vane test was integrated to the proctor test. After the mould was weighed, each time a hand vane device was penetrated into the compacted soil, to measure the shear strength. However, for the first 2 or 3 repeats of the tests, the hand vane device could not be penetrated due to stiffness of the soil sample, because of having low moisture content. For the last several repeats, it could be penetrated but for some of them its exact shear strength still could not be measured because it was so high that the capability of the device was not sufficient to measure. The device could only measure the shear strength up to 150 KPa. In some tests, the soil did not shear although the device showed 150 KPa. That meant the soil at that moisture content had shear strength of more than 150 KPa. However, the fifth repeat of each test could give a clear shear strength value.

The device was penetrated at one point in each layer. 3 values were obtained from each test repeat when possible. The strength of the bottom layer was always highest, and then the second was middle layer and the third was the top. Because, the bottom layer was subjected to the stress of the upper layers and it was affected by the blows given to the other layers while compacting. An average of those was calculated and noted as shear strength value.

6.5.3 Calculations

The results were plotted in the same graph as compaction. They could give an indication of shear strength to compare the results with the ones from triaxial test. A photo from the shear vane test is shown in figure 23 and the results are represented below. Only one sample's results are shown in detail below and the others in the appendix to ensure the clarity of the dissertation. However, the results obtained in the last repeats of the each test are gathered in one table and shown below. As seen from the results, shear strength of the soil is decreasing when moisture content increases.

Figure 23 Shear vane apparatus penetrated into soil

Table 13 Dry density – moisture content – shear strength relation of TP3B1

Vane shear test results (hand aparatus)					
TP3B1	Test 1	Test 2	Test 3	Test 4	Test 5
Dry density (Mg/m3)	1.28	1.37	1.35	1.27	1.18
Mositure content (%)	24.63	29.75	35.48	40.17	44.59
Shear strength at centre (kPa)	>150	>150	120	50	20
Shear strength at point 2 (kPa)	>150	>150	120	44	20
Shear strength at point 3 (kPa)	>150	>150	>150	65	20
Average shear strength	>150	>150	130	53	20

Table 14 Dry density – moisture content – shear strength relation for the last repeats of the tests

Vane shear test results (hand aparatus)				
Last repeat of the tests	TP1B2	TP2B2	TP2B3	TP3B1
Dry density (Mg/m3)	1.19	1.28	1.32	1.18
Mositure content (%)	45.28	38.70	36.19	44.59
Shear strength at centre (kPa)	80.00	105	112	20
Shear strength at point 2 (kPa)	72.00	86	85	20
Shear strength at point 3 (kPa)	81.00	119	121	20
Average shear strength	77.67	103.33	106	20

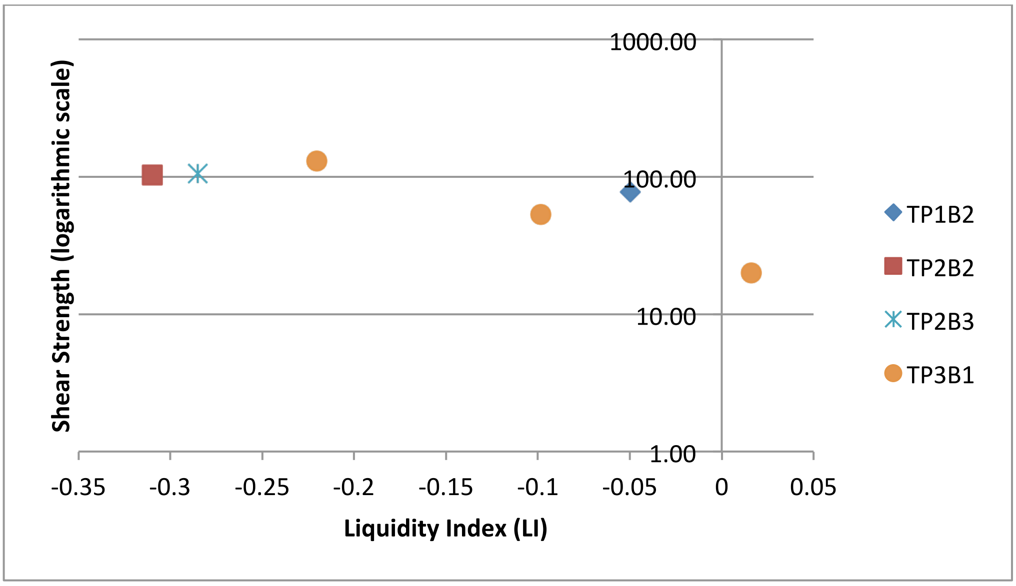


Figure 24 Shear Strength-Liquidity Index Relation Obtained from Vane Test

6.6 Triaxial Compression Test

6.6.1 Scope of the Test

Triaxial compression test is included in this study to determine the shear strength parameters of soil. The purpose of the test is finding cohesion and behaviour of the soil under axial load. In this test, a cylindrical specimen of soil is subjected to an axial major principal stress σ_1, and a radial minor stress σ_3 as represented in figure 25. Diameter of the specimens was 38 mm as it is stated by BS 1377, Part7; specimens of 38 mm diameter are suitable for homogeneous fine-grained cohesive soils. The height of the specimen was 76 mm as it should be around twice as the diameter stated by the standard.

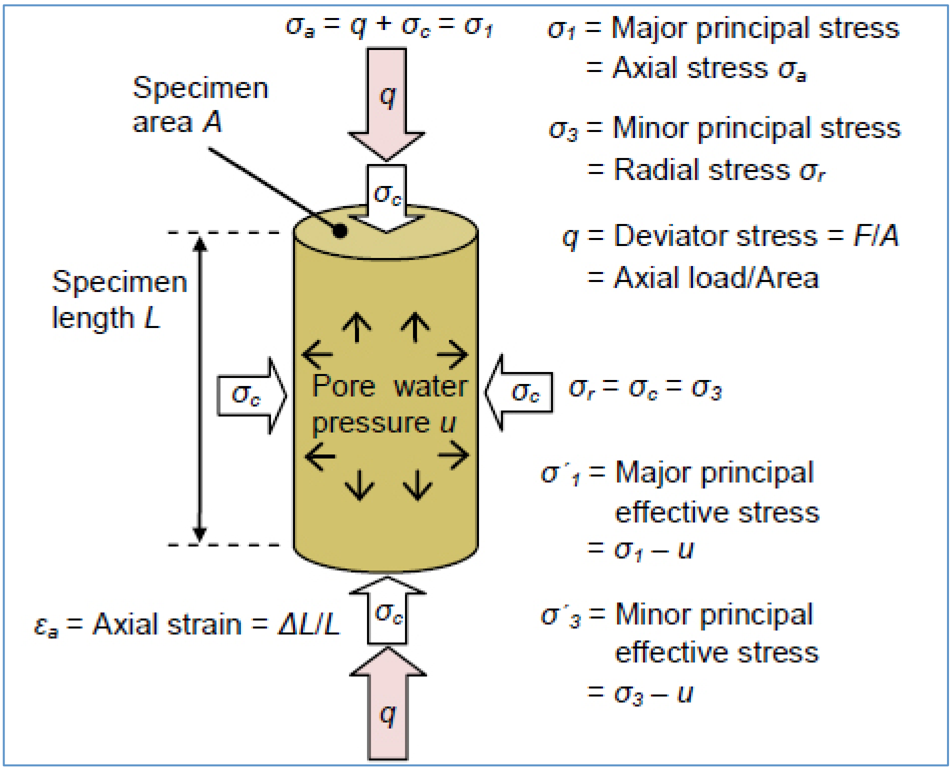


Figure 25 Stresses on cylindrical specimen subjected to triaxial compression (Rees, 2013)

Triaxial test has some advantages comparing to simpler shear strength tests like direct shear strength test. It is possible to control specimen drainage and to take measurements of pore water pressures, with triaxial tests. Principal parameters can be obtained from triaxial test including the angle of shearing resistance ϕ´, cohesion c´, and undrained shear strength cu, besides other parameters like the shear stiffness G, compression index Cc, and permeability k may also be determined. However, this study did not involve those other parameters. An example of the engineering application of the test is demonstrated in figure 26. It shows that triaxial compression provides information on the strength of the upper part of the cut slope, while triaxial extension enables to determine parameters for soil elements at the slope base. (Rees, 2013)

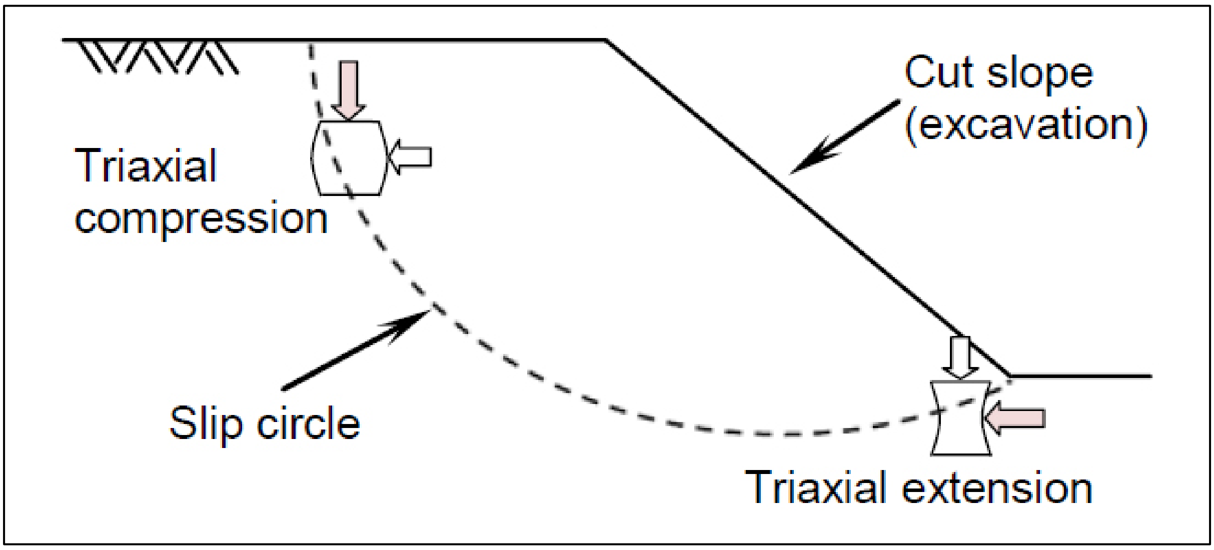


Figure 26 Example of an engineering application of the triaxial test (Rees, 2013)

6.6.2 Procedure of the Test

Triaxial test was done on 4 samples, namely TP1B2, TP2B2, TP3B1, TP4B2, which are the bottom samples of 4 trial pits. For each test, 3 standard specimens were prepared at the optimum moisture content, they were weighed and noted. Then, they were put into the cell after covering with a 0.2 mm thick membrane. For the first specimen, 50 KPa lateral pressure was applied. It was 100 KPa for the second, and 200 KPa for the third. Machine speed was adjusted to 1 mm/min. When everything is set, tests started.

The readings were taken until it reached a constant load. The shear of the soil was watched during the test. After the test finished, the water in the cell was evacuated, the specimen was taken and the membrane was removed. Photos of the sheared soil samples were taken for each specimen. A representative sample was taken to measure the moisture content for each specimen, weighed, noted and put into oven to dry. The next day, the samples were taken from the oven and weighed, the moisture content was determined.

6.6.3 Calculations

After finishing the tests, data of vertical load and deformation of the sample was obtained. From these data, deviator load and axial strain was calculated and tables were made. Then, graphs were plotted using the values of deviator stress in the ordinate, against axial strain in the abscissa. After that, Mohr circles were drawn to obtain the angle of shearing resistance ϕ' and cohesion c'. Since the tables of readings are too long (about 2500 rows of excel for each specimen) they are not represented. However, overall results of all the samples are in the appendix. Results of only one test are shown below and the others in the appendix, to ensure clarity of the dissertation.

Table 15 Specifications of samples and maximum deviator stress results for samples of TP2B2

TP2B2	Sample 1 = 50kPa	Sample 2 = 100kPa	Sample 3 = 2000kPa
Weight	152.82	154.16	152.36
Length of sample (mm)	76		
Diameter of sample (mm)	38		
Area of sample (mm2)	1134.11		
Horizontal stress (kPa)	50	100	200
Revolution of reading	1mm/min		
Max Deviator Stress (Kpa)	487.13	653.05	821.32

Table 16 Moisture Content for triaxial test on TP2B2

Moisture content determined on the sample at failure (TP2B2)			
Container No:	TP2B2-1	TP2B2-2	TP2B2-3
Wet soil + container (g)	51.9	51.09	59.3
Dry soil + container (g)	42.84	42.53	50.41
Mass of container (g)	14.72	14.04	22.28
Water content (%)	32.21906	30.04563	31.60327
AVERAGE MOISTURE CONTENT	**31.29**		

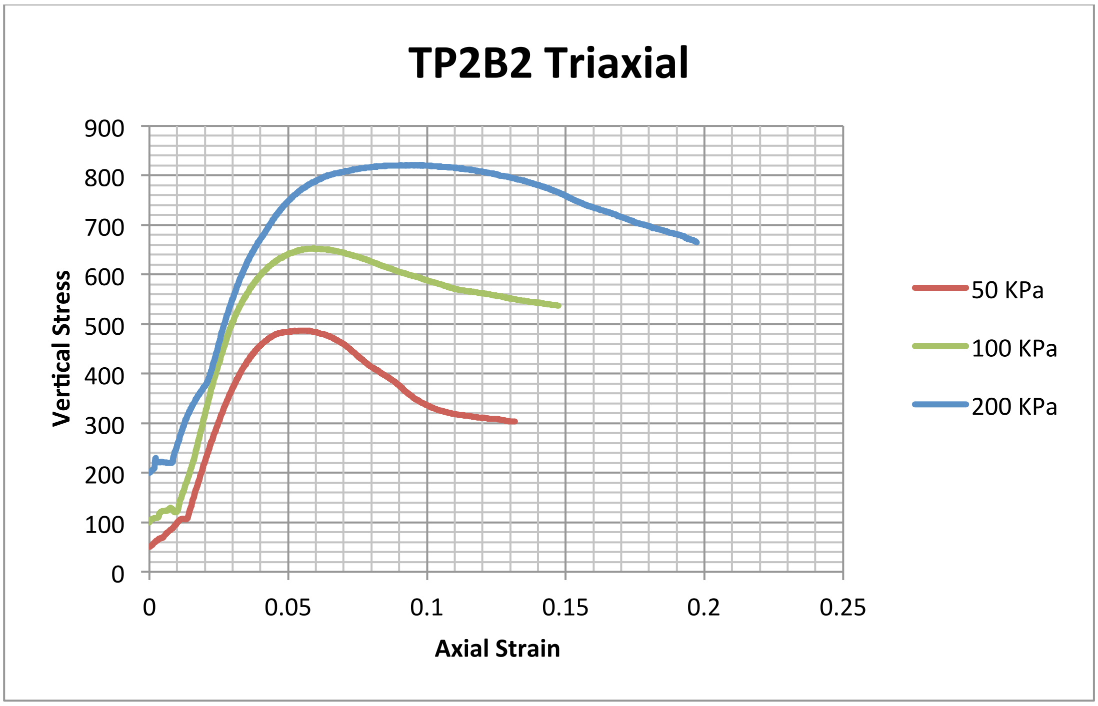


Figure 27 A typical Axial Strain – Vertical Stress Relation of Mercia Mudstone obtained from TP2B2

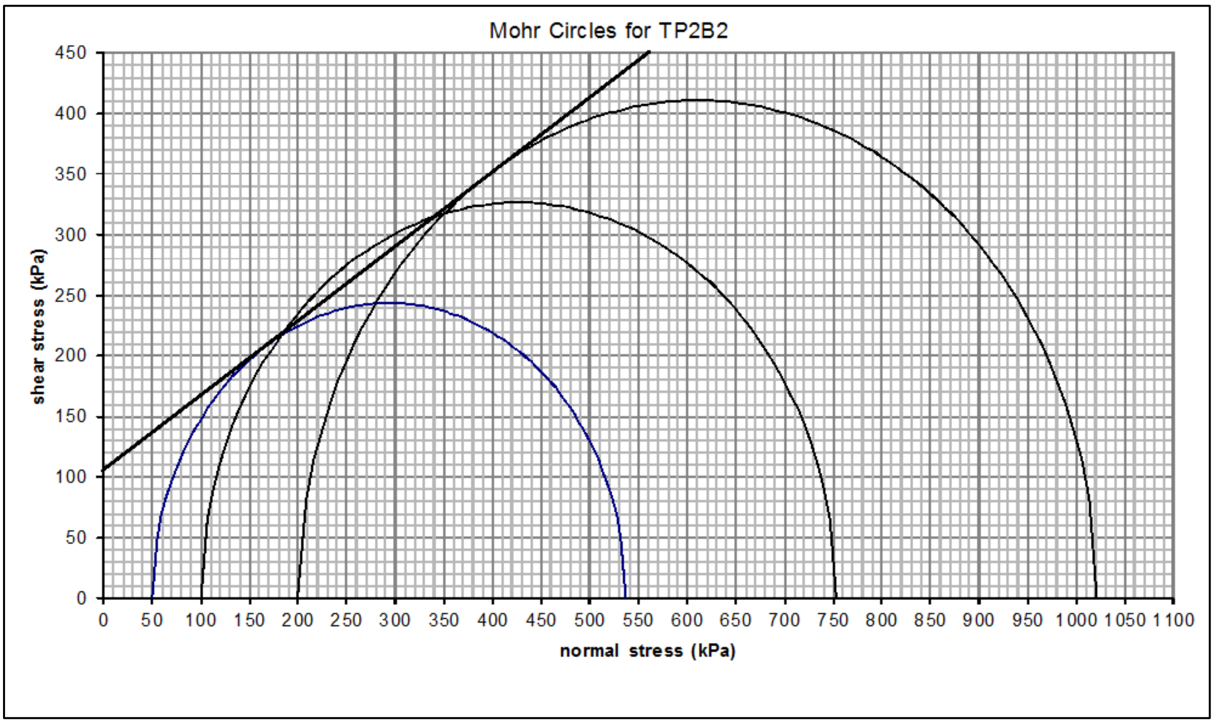


Figure 28 A typical normal stress – shear stress diagram of Mercia Mudstone sample from TP2B2

After Mohr circles are drawn, shear strength of the sample was calculated. To calculate the shear strength, the shear strength envelope was fitted to the circles. The Mohr envelope was selected to be a close enough approximation to a straight line tangential to the curves. It does not have to touch all the circles as long as it is the best fit for all the circles and forms a straight line. After the shear envelope was drawn, the angle of shearing resistance ϕ′, cohesion c′ and the equation of the envelope were obtained. Basically, the angle of shearing resistant ϕ′ is the angle of the Mohr envelope with the horizontal. The cohesion c′ is equal to the value which corresponds to the point of intersection of the mohr envelope and shear stress axis. Then, the equation of the envelope can be formed. The equation is:

$$\tau = c' + \sigma.\tan\phi'$$

If a line is drawn from the center of the circle in the middle, having an angle of (90+ ϕ′) to touch the envelope, then shear strength at failure can be calculated as shown in figure 29.

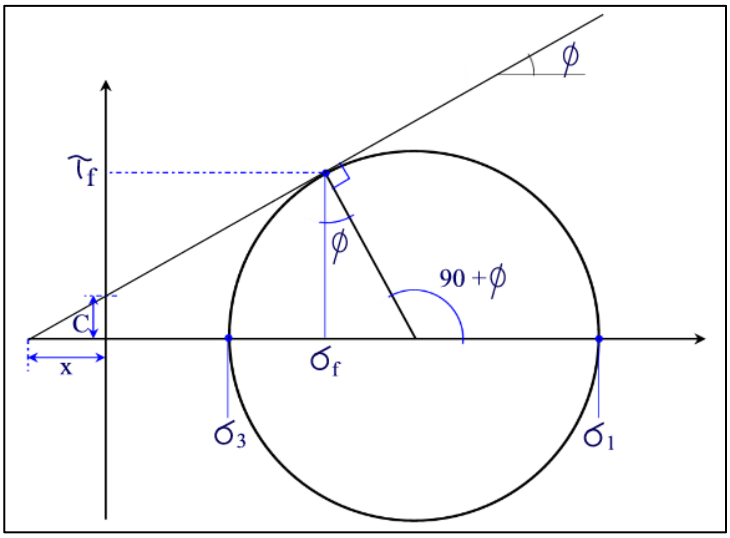


Figure 29 An example of shear strength calculation from Mohr's envelope

Results for c´ and ϕ´ are shown in table 17 for all samples.

Table 17 Results of Triaxial Tests

Sample	Moisture Content	c´ (Kpa)	ϕ´ (°)
TP1B2 (0.75 m)	37.96	70	41
TP2B2 (0.55-0.65 m)	31.29	107	37
TP3B1 (0.6 m)	29.02	60	42
TP4B2 (0.8 m)	36.61	64	42

7 Design

In this chapter, design of the proposed construction is analysed. The results of the tests are evaluated, a proper design according to the standards, using the data from tests is tried to be managed. Previous design by Lamb (2013) is analysed and improved. It is tried to select a location of the rail line which is most proper in terms of capability of the ground, minimum earthwork and satisfying drainage. Network Rail standards are followed to ensure safety of the construction although it does not have to be followed since it is an independent private work. However, it is good use of standards as the Network Rail standards aim to ensure a high quality design providing financial efficiency of delivering the work and whole life cost, minimising the health and safety risk though specification and design (NR/L3/TRK/6002)

7.1 Drainage

Drainage is one of the essential parts of railway infrastructure. Problems related to water such as wet-beds arising are prevented by convenient drainage systems. As noted by Network Rail standards (NR/L3/CIV/005/1), drainage systems control the water effectively to ensure safe and economic management of infrastructure. The main sources of water can be precipitation on the track, run off from areas next to the track and groundwater from underlying permeable layers, perched water tables and infiltration through the ballast and ditches. If railway drainage is not designed properly, it may cause temporary closures of line, as well as increase in maintenance cost. Chabot, Sandrone and Gamisch (2013) also agreed that selecting suitable design and proper materials as well as maintenance approaches is essential to prevent problems and to decrease maintenance costs.

As noted by Rushton and Ghataora (2009), Railway track drainage has a very important influence on both the behaviour of rail against loading and the durability of the ballast and sub-ballast. There are numerous standards and design methods for the surface drainage however there is not much information about sub-surface drainage. Also, although mathematical analysis of drainage for agricultural land and for civil engineering structures has a quite long history; there is not any detailed study about the drainage of railway ballast has been identified. (Rushton K. and Ghataora G., 2009)

Drainage itself is not a specific component; it is formed of series of components. Basically the components of drainage include ballast and drains.

Ballast is one of the most important components of railways as well as it is significantly important to railway drainage. Besides its main structural function, it is also needed to drain precipitation falling on the track. According to Railway Group Standard (GC/RT5021), the required depth of ballast layer depends on the desired life before it needs to be replaced, the proposed maintenance method, type of the sleeper, the existence of a geosynthetic beneath the ballast, and site constraints.

The specifications of drainage components are described in Network Rail Standards (NR/L3/CIV/140/52C) and the design life of the materials used for drains is determined to be at least 60 years. In the standard, the outlet pipes are also told to be 150mm in diameter and located at 100 metre intervals or at specific low points.

However, Network Rail standards are prepared for major railway projects which are managed by Network Rail, and the Midsomer Norton siding project is a short in terms of track's length and a basic heritage railway project, the standards does not have to be strictly followed.

Maps of Environment Agency are checked. It is found that there is a minor risk from rivers near the site. However, the site is on the top of the hill above the river. Also, there is no risk detected from any reservoir near the site. So, the probability for flooding in the area of the site is very low. Maps were taken from website of Environment Agency, presented in figure 30.

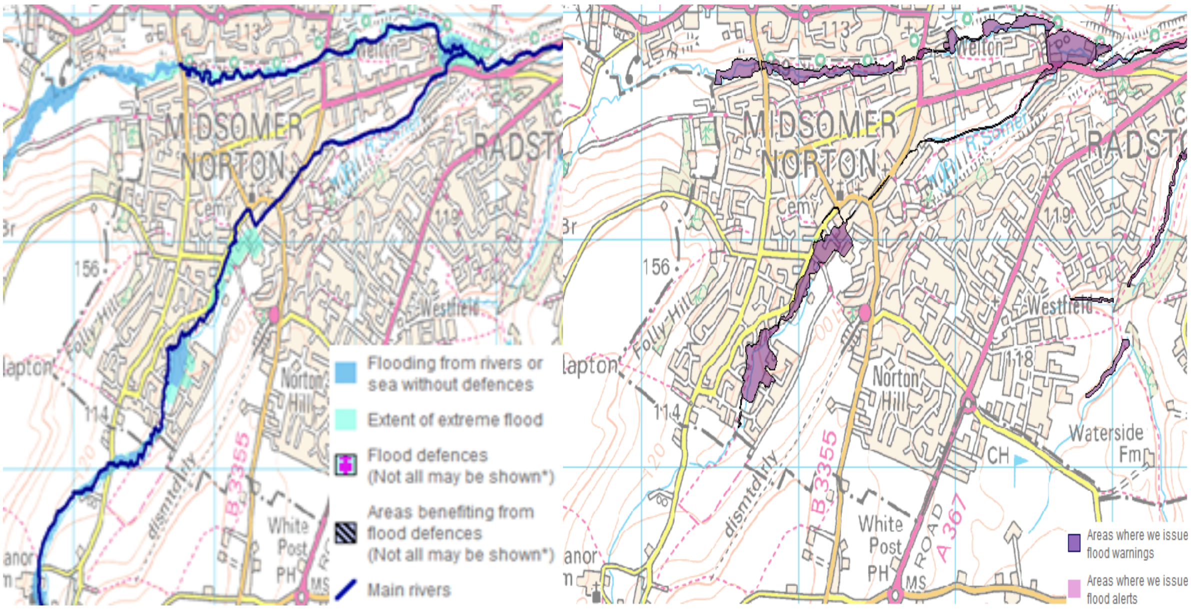


Figure 30 Flood Risk from Rivers and Flood warning Maps (maps.environment-agency.gov.uk)

Although there is no flood risk near the area of the site, the moisture content of the ground is found to be between 28-41 %. So it can be admitted that the moisture content is not low.

There is already a drainage system for the Midsomer Norton railway, adjacent to the proposed location of siding. As mentioned by Lamb (2013), drainage is provided by an internal drainage network and out of the retaining wall near the station to drain from the railway and the retaining walls. Also, the public drainage system is present for the local roads nearby the sidings, which is managed by the local council.

If it is ensured that the existing drainage system works efficiently, there might not be a need for a detailed drainage system for the sidings. However, a well-drained trackbed is essential to good track maintenance.

When the earthwork of the sidings is considered, if cutting is built with a high slope, it might be seen necessary to build a retaining wall between the siding and the railway which brings extra need for drainage system.

It is recommended to build a fin drain on the side of the railway siding, and an outlet pipe in the middle of the siding, to remove any risk of flood and to protect the ground from landslide because of high moisture content of the ground.

7.2 Earthwork

There are many factors that may affect the stability of a site which is proposed for earthworks. In the natural conditions, the site can be unstable. Also, there is a probability that building cutting or embankments is likely to cause instability of underlying or surrounding ground, even if it was in a stable condition before. (BS 6031, 1981)

The ground in the site of proposed construction is stable for now, as there is no load due to any building and there is no landslide noted in the history. However excavations for cutting will affect the stability of the ground. So, proper compaction to the embankments and a safe slope angle is needed to apply. This chapter covers the final design of the sidings, evaluating the test results and trying to minimise the earthworks. As the geology of the site is very important for the earthworks, all relevant information was summarised in maps drawn to a suitable scale using Autocad.

The design must ensure safety and there must not be any risk of failure. As noted by BS 6031 (1981), the risk of failure should be considered under two headings. First is the movement due to failure of the ground in shear, and the second is unacceptable deformation before failure is reached. In order to evaluate the risks of failure in shear of the earthworks, a safety factor is used. Safety factor is principally described as the ratio of the available shear strength of the soil to the strength needed to maintain equilibrium. The properness of value of the safety factor is associated to the consequences of failure. For cuttings, a high safety factor is used because a failure of cutting is likely to cause damage on the railway and would be very dangerous to life. Embankments are also considered in the same way.

Considering the economy of the project, the steepest possible slope enables the minimum cost of the cutting. However, noted by BS 6031 (1981), the economic and human consequences of failure of a cutting need careful consideration.

When making an assessment of stability conditions, the soil classification obtained from laboratory tests, such as natural moisture content test, liquid limit test, plastic limit test gives an indication of the behaviour of a particular type of soil when excavated from a cutting. Also, the compaction and shearing resistance parameters obtained from the other laboratory tests are used to analyse the stress changes which take place in the material, both in the short and long period, as a consequence of excavating for the cutting. (BS 6031, 1981)

Both the height of the slope and the shearing resistance of the soil are taken into account when the stability of a slope in a cohesive soil. When the cutting is needed to be higher, then the slope angle has to be flatter. For normally consolidated clays, as noted by BS 6031 (1981), it is possible to determine the short term stability using the results of simple shear strength tests. However, strength of the soil reduces as a result of stress consequent on excavation for the cutting, therefore the slope may fail by a rotational slide in the medium or long term even it is stable in the short term.

While doing the design, following issues were considered:

- The design must provide a good track alignment
- It must ensure a good slope stability
- Construction costs must be minimised.

Once all these are covered, a proper design is processed.

7.2.1 Alignment Design

An alignment was drawn in the boundaries of the proposed construction area. Based upon the previous study by Lamb (2013), a similar location was selected for the minimum earthwork. The curve which is leaving the mainline has a radius of 100 m, as the curve connecting the flat section of the siding to the part coming from the mainline has a 55 m radius. As trains will slow down in this section of the track, it is not seen necessary to build a curve with a bigger radius.

First, the alignment was drawn and the longitudinal profile was plotted. While drawing longitudinal profile, the middle of the track was taken as reference, and heights of the points on the middle line were determined by interpolation. From the points which had been marked with their height in the level survey map, lines were drawn to the reference line, and the heights of the points on the line were calculated according to the distance between the line and the marked points.

As seen from the longitudinal profile in figure 32, it is apparent that there is a big demand for fill. However, the amount of earthwork is decided after cross-sections are drawn. Locations of the cross-sections and the longitudinal profile are represented in the figures below.

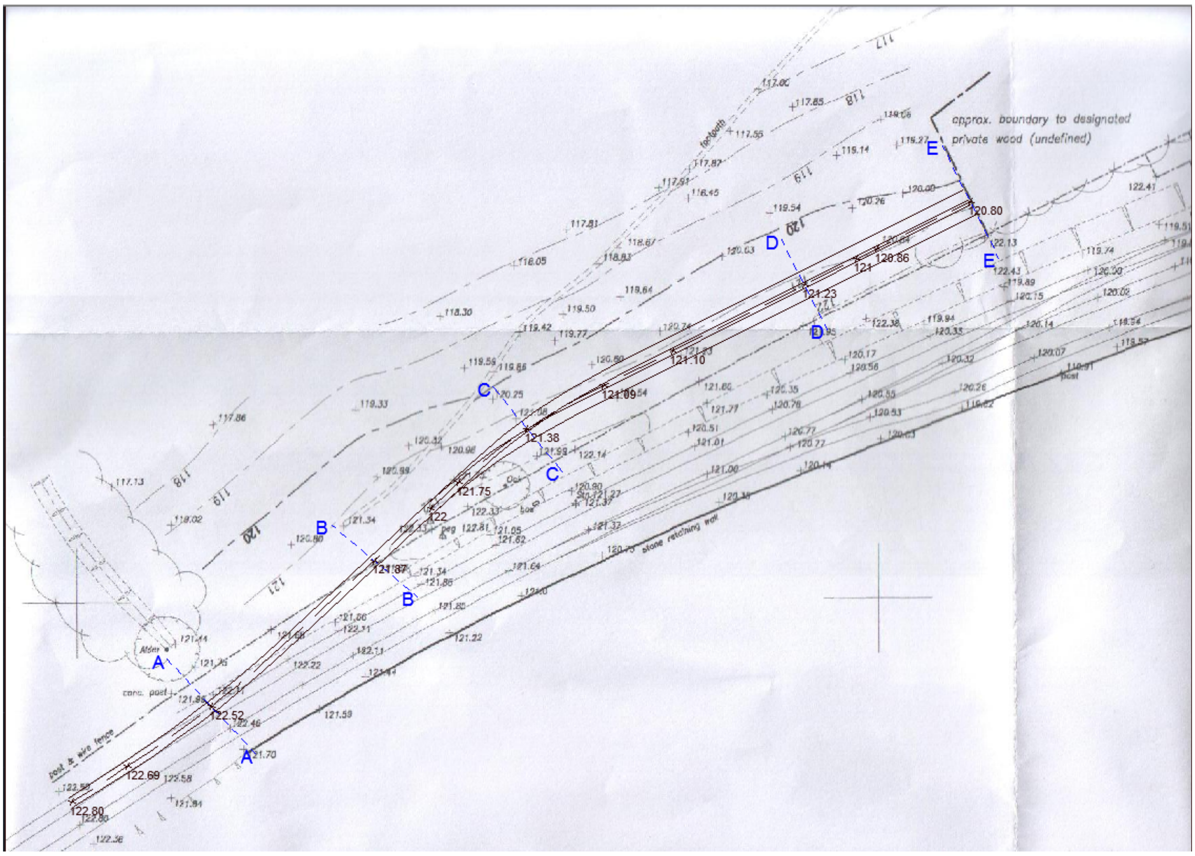


Figure 31 Layout of the alignment and locations of the cross-sections

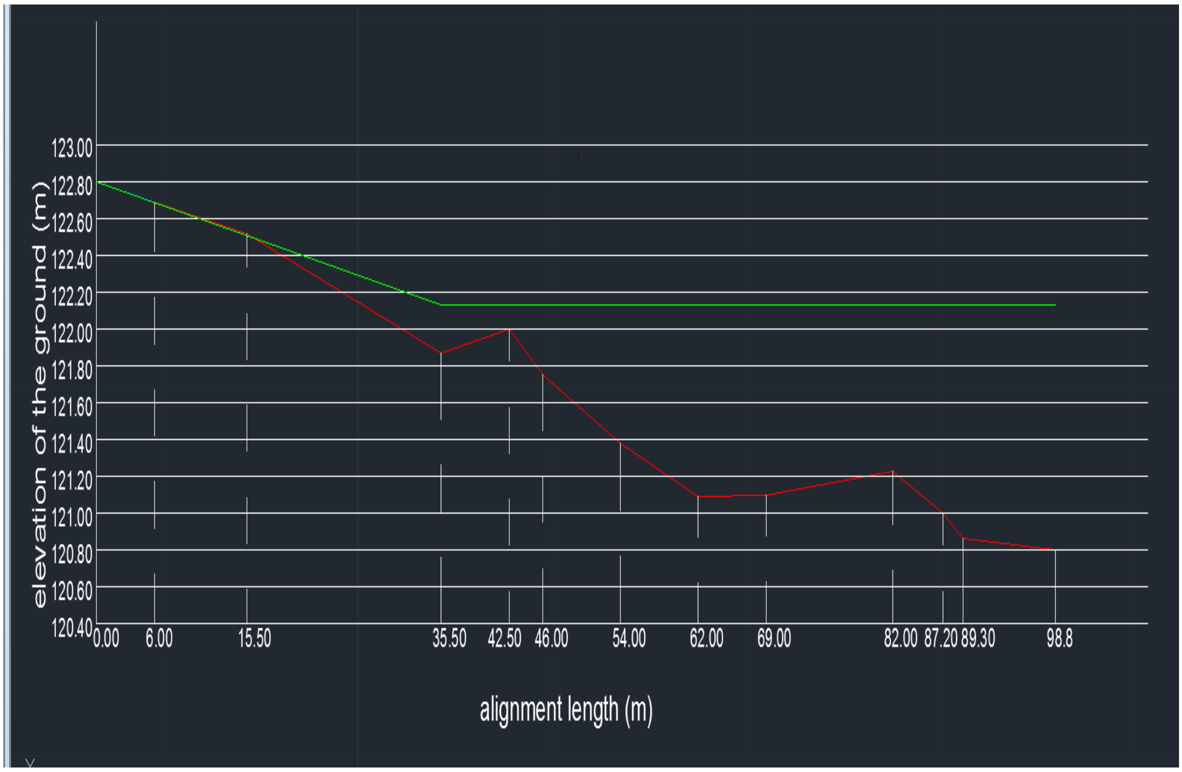


Figure 32 Longitudinal Profile of the proposed alignment (red line shows ground and green line shows elevation of the proposed alignment)

The gradient was kept same as the mainline railway at the beginning section of the siding as 1:53, and then it was made flat at the last 60 m section of the track. After longitudinal profile, cross sections were taken from determined positions, cut and fill areas were calculated. After that, total volume of the earthworks was calculated.

Five points were selected for plotting cross-sections. Areas of fill were calculated. After areas were obtained, total needed fill volume was calculated. Cutting area for cross-section A-A is 0.6348 m^2, for B-B is 1.2152 m^2, for C-C is 5.0727 m^2, for D-D is 6.0606 m^2 and for E-E is 10.3196 m^2. So, total volume of earthwork was found to be 370.123 m^3 of fill.

7.2.2 Slope stability and Bearing Capacity

It must be ensured that the cut and fill slopes are stable, and the constructible embankment height does not exceed the maximum.

As derived from Look (2007), the ideal cut slope for clay type soil is between 1:6 and 1:2 (Vertical : Horizontal) and for fills, typical batters of slopes vary between 1:4 and 1:1.5 (Vertical : Lateral). So, it was seen ideal to have a 1:2 (vertical : lateral) slope for the cuttings and 1:1.5 slope for fills as it is also in the limits of british standards. Tables adopted from Look (2007) are presented below.

Table 18 Typical values for cutting slopes (Look, 2007)

Material	Slope batters (Vertical : Horizontal)	
	Permanent	Temporary
Massive rock	1.5V: 1H to Vertical	1.5V: 1H to Vertical
Well jointed/bedded rock	1V: 2H to 2V: 1H	1V: 2H to 2V: 1H
Gravel	1V: 2H to 1V: 1H	1V: 2H to 1V: 1H
Sand	1V: 2.5H to 1V: 1.5H	1V: 2.5H to 1V: 1H
Clay	1V: 6H to 1V: 2H	1V: 2H to 2V: 1H

Table 19 Typical values for fill slopes (Look, 2007)

Material	Slope batters (Vertical : Horizontal)
Hard rock fill	1V: 1.5H to 1V: 1H
Weak rock fill	1V: 2H to 1V: 1.25H
Gravel	1V: 2H to 1V: 1.25H
Sand	1V: 2.5H to 1V: 1.5H
Clay	1V: 4H to 1V: 1.5H

According to these values of slopes for cut and fill, cross sections were drawn and stability analysis were done.

After the cross-sections were drawn, the first step of the slope stability analysis is calculating P_d which is calculated as noted by (Duncan and Wrirght, 2005):

$$P_d = \frac{\gamma H + q - \gamma w H w}{\mu q \mu w \mu t}$$

Where

γ : average unit weight of soil (F/L3)

H : slope height above toe (L)

q : surcharge (F/L^2)

γw : unit weight of water (F/L^3)

Hw : height of external water level above toe (L)

μq : surcharge reduction factor

μw : submergence reduction factor

μt : tension crack reduction factor

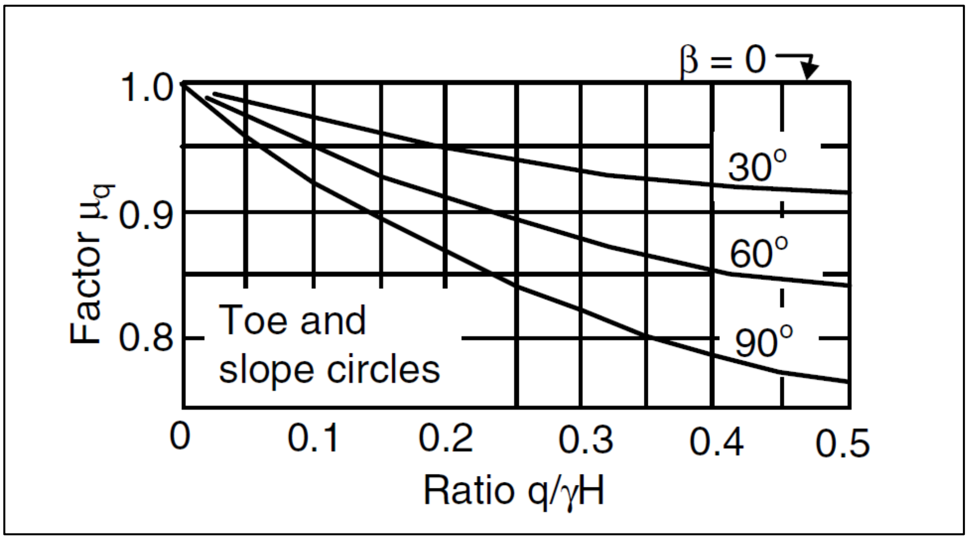


Figure 33 Surcharge Adjustment Factors (Janbu, 1968)

Surcharge on the soil is caused by train load, therefore it is calculated as the maximum train load divided by the area on which this load will be distributed. It is assumed that the maximum train weight will be 135 tonnes and 20 in length, which will be distributed on an area of 4 m width subgrade. Surcharge is calculated as 16.875 KPa which is taken 17 KPa in the further calculations to be on the safe side.

Surcharge reduction factor µq is found 0.92, obtained from the surcharge adjustment factors chart which is represented in figure 33; µw is assumed to be equal to 1 as there is no external water above the toe, and µt is assumed to be equal to 1 as there are no tension cracks. So, our equation becomes:

$$P_d = \frac{\gamma H + q}{0.92}$$

γ was taken as 18 kN/m^3 as the average density of the samples obtained from laboratory tests. Maximum height of fill was found to be 2,65 m. Therefore Pd is 70.33 kN/m2.

The next step is calculating dimensionless parameter $\lambda_{c\phi}$ to obtain N_{cf} from the slope stability chart developed by Janbu (1968).

$$\lambda c\phi = \frac{Pd.\tan\phi}{c}$$

where ϕ is the average value of shearing resistance angle and *c* is the average value of cohesion intercept. As a design parameter, average of the results of shearing resistance angle is taken as ϕ = 41°, and average of cohesion intercept is c = 75 Kpa which were obtained from laboratory tests.

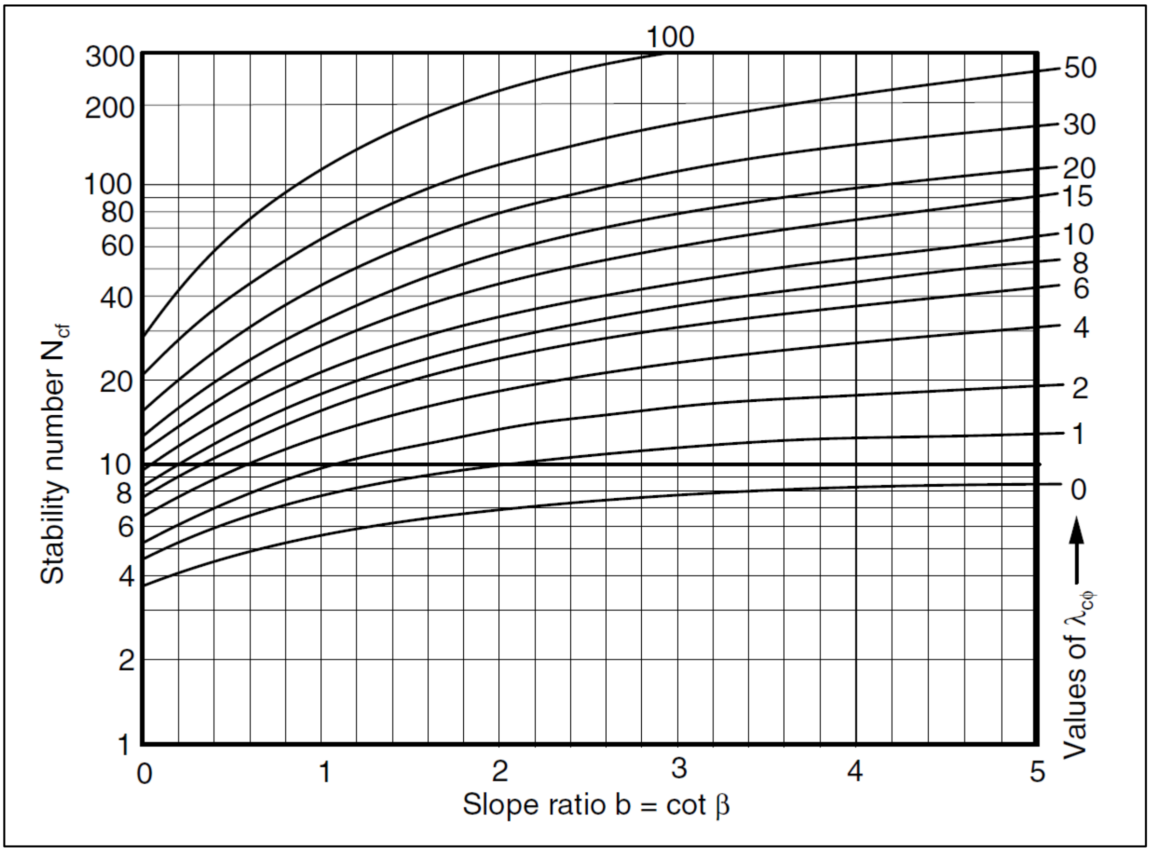


Figure 34 Slope Stability Chart for ϕ > 0 soils (Janbu, 1968)

So, $\lambda c\phi$ is equal to 0.815. N_{cf} was found 8 from the chart. Then safety factor was calculated as:

$$\mathrm{F.S} = \mathrm{N}cf\,\frac{c}{P_d}$$

Safety factor was found 8.53. It is much higher than 1.5. So the ground stability checks okay.

It is planned that 4 locomotives with a maximum weight of 135 tonnes for each will be supplied in the sidings. It is assumed that sidings will be used for storing rolling stock in the long term, and that the case of a maximum loading should therefore be considered. On the first siding nearest the main line, this is likely to be used for locomotives. The heaviest locos are estimated to be about 135 tons, and four could be accommodated along siding, so a maximum loading of about 550 tons is considered to be the maximum. This figure is unlikely to be reached, but should still be considered as a control loading.

Bearing capacity of the soil was calculated according to the equation developed by Terzaghi, represented below:

$$q_f = s_q N_q \sigma'_q + 0.5\gamma B s_\gamma N_\gamma + S_c N_c c'$$

Where N_q is a bearing capacity factor relating to surcharge applied around a foundation under drained conditions and s_q is a shape factor. N_γ is the bearing capacity factor relating to self-weight, N_c is the factor relating to cohesion, and s_γ and s_c arefurther shape factors. (Knappet and Craig, 2012)

Nq is calculated as $Nq = \frac{(1+\sin\phi')}{(1-\sin\phi')}e^{\pi\tan\phi'} = 74$

N_γ is calculated as $N_\gamma = (Nq-1)\tan(1.32\phi') = 101$

Nc is calculated as $Nc = \frac{Nq-1}{\tan\phi'} = 84$

So, bearing capacity q_f was calculated and found to be 7663.5 KPa, which is much higher than what is proposed to be supplied on the track.

7.2.3 Compaction

The relationship between dry density and moisture content was established by standard compaction test applied on 4 samples from different pits. It gives the idea of how much the embankments should be compacted when filled during construction. Also, noted by Williams (1949), usually field compaction trials are processed in addition to laboratory tests by using the actual placing and compaction equipment which is to be used for construction. By doing this, the results of the laboratory tests are checked in the site, accuracy of the tests are confirmed.

By doing compaction test on the deepest samples from 3 trial pits (TP1, TP2 and TP3), optimum moisture content was found to be between 31.2% and 37.4%, and maximum dry density ranges from 1.25 to 1.38 g/cm^3. Based on the results of the compaction test, the soil should be compacted to a sufficient degree to ensure stability of the embankments. Also, additional care must be taken for overcompaction. Because when soil is compacted more than a sufficient degree, for especially fine-grained soils, it causes soil easily absorb water leading swelling, reduction in shear strength, and increase in compressibility. It is noted by Head (1980), toes and sides of embankments are especially sensitive to this effect.

British standards (BS 6031, 1981) recommend several types of compaction plant including sheepsfoot roller, smooth wheeled roller, pneumatic tyred roller, vibrotary roller over 70 kg per 100 mm roll, vibratory plate compacter over 1400 kg/m^2 of base plate, vibro-tamper and power rammer for fine soils. However, it is also noted that sheepsfoot roller is the best option for the soils having natural moisture content below their plastic limit. As our soil has a natural moisture content value of lower than its plastic limit, it is recommended to use sheepsfoot roller for compacting the soil. Depending on type of plant, it is also noted minimum number of passes for satisfactory compaction ranges from 4 to 8, and the maximum thickness of compacted soil layer from 100 to 450 mm.

Number of passes of the roller also depends on the height of the fill. A table is represented by Look (2007) and demonstrated below, showing an example of such a variation, assuming similar materials being used throughout the full height. It is also agreed by Look (2007) that the optimum compaction thickness is dependent on the type of equipment used.

Table 20 Typical number of roller passes needed for 150mm thick compacted layer (Look, 2007)

Height of fill (m)	Number of passes of roller for material type		
	Clayey Gravel (GC)	Sandy Clay (CL), clayey sand (SC)	Clay, CH
<2.5m	3	3	4
2.5 to 5.0 m	4	5	6
5.0 to 10.0 m	5	7	8

As our material is in the group of Clay, CH, and maximum height of fill is slightly more than 2.5 m; 4 passes of a sheepsfoot roller would be enough for 150 mm thick compacted layer.

8 Conclusions

8.1 Findings

In this study, a geotechnical ground investigation is issued using systems engineering approach. Feasibility analysis are performed for the proposed railway sidings and the depot at Midsomer Norton Station of Somerset and Dorset Railway.

The soil material found on the site is Mercia Mudstone. Laboratory testing results showed the characteristics of the soil taken from the site. Average moisture content was found to be 35 %. However, moisture content depends on the location and depth of the sample. It was seen that the moisture content increases with depth.

Liquid Limit and Plastic Limit tests were carried out to be able to achieve a classification of the soil. Liquid limit ranges from 63.66% to 90.13% and plastic limit is between 37.92% and 54.67% depending on the location of the sample. These results demonstrated the soil is a type of very high plasticity clay.

Standard compaction test was processed to know the relation of optimum moisture content with maximum dry density of the soil. Average of optimum moisture contents (33.55%) and maximum dry density values (1.34 g/cm^3) indicate how much the soil should be compacted.

Vane tests and triaxial compression tests were performed to obtain the soil strength. Average cohesion was found c' = 75 Kpa and average shearing resistance angle $\phi = 41°$.

These parameters allowed to analyse slope stability and bearing capacity of the soil. Based upon the laboratory test results, all stability analysis were done and ground was found capable to build the proposed structures.

An alignment was drawn and amount of earthwork was calculated using Autocad. It was found that there is a need for 370.123 m^3 of fill material.

When the design was processed, the left hand side of the V – diagram used in systems engineering was completed.

8.2 Recommendations

It is strongly recommended that systems engineering approach should be followed in the construction stage, as well as the design stage, to provide traceability, and to ensure dependability. As railways are expected to operate with increasing levels of reliability, availability, maintainability and safety; the use of systems engineering is one way of ensuring high levels of dependability.

There is a need for a scaled level survey map to perform a precise design of the earthworks, as the existing map is so empirical. So, an improved geological map needs to be obtained in order for the calculations to be more accurate.

Also, an archaeological survey needs to be undertaken to know the archaeological conditions of the underground. If anything antique or historically valuable is found in the site after building the track and depot, it might be needed to remove the construction and excavate the ground beneath. That means in such a case, all the work done will go for nothing.

It is recommended to analyse the performance of existing drainage systems and to obtain rainfall statistics to cover exact design of the drainage, after determining the certain location of the siding.

It is also recommended to do further laboratory tests including sulphate content and consolidation tests to confirm the validity of the design performed in this study.

8.3 Review of Approach

Conducting a systems engineering approach is a good use of this study, as it provides traceability for all stages of the design, and therefore helps any problem in the design and implementation to be solved more easily.

8.4 Areas for Further Work / Research

As this study covers the ground investigation and design of the substructure, a further study is needed on the superstructure construction design to complete the building process of the siding and the depot.

8.5 Word Count

There are 14753 words between sections 1 and 8.5.

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9.2 Web Site Links

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Appendix A - Test Results

Moisture Content Test Results

Moisture Content								
Location: Midsomer Norton, Bath and North East Somerset, UK					Job Ref: Usame Midsomer Norton			
Soil Description: Mercia Mudstone								
Test Method: BS 1377: Part 2: 1990: 3.2					Date: 25.07.2013			
Sample Number	TP1 B1	TP1 B2	TP2 B1	TP2 B2	TP2 B3	TP3 B1	TP4 B1	TP4 B2
Mass of container (m_1)	392.9	497.0	348.5	394.4	393.7	393.7	391.6	392.1
Mass of wet soil + container (m_2)	1116.0	1340.8	1282.7	1117.8	1131.2	1164.1	1207.1	1090.3
Mass of dry soil + container (m_3)	957.9	1116.8	1065.0	936.8	927.4	964.6	981.7	888.8
Mass of moisture (m_2-m_3)	158.1	224.0	217.7	181.0	203.8	199.5	225.4	201.5
Mass of dry soil (m_3-m_1)	565.0	619.8	716.5	542.4	533.7	570.9	590.1	496.7
Moisture Content w = [(m_2-m_3)/(m_3-m_1)]*100 %	28.0	36.1	30.4	33.4	38.2	34.9	38.2	40.6

Liquid Limit Test Results

TP1 B1												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	16.1	16.0		19.0	18.7		21.9	21.3	20.9	22.3	22.1	
Average Penetration	16.05			18.85			21.37			22.20		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	13.20			13.56			14.13			13.51		
Mass of dry soil + container (g)	8.86			9.18			8.90			8.52		
Mass of container (g)	3.72			4.11			3.23			3.25		
Mass of moisture (g)	4.34			4.38			5.23			4.99		
Mass of dry soil (g)	5.14			5.07			5.67			5.27		
Moisture content (%)	84.44			86.39			92.24			94.69		
Liquid Limit (%)	90.13											

TP1 B2												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	17.1	17.1		19.1	19.4		22.4	23.0	22.3	23.2	22.5	23.4
Average Penetration	17.1			19.25			22.57			23.03		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	12.68			14.05			14.88			13.25		
Mass of dry soil + container (g)	8.58			9.41			9.27			8.40		
Mass of container (g)	3.72			4.11			3.23			3.25		
Mass of moisture (g)	4.10			4.64			5.61			4.85		
Mass of dry soil (g)	4.86			5.30			6.04			5.15		
Moisture content (%)	84.36			87.55			92.88			94.17		
Liquid Limit (%)	88.94											

TP2 B1												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	16.4	16.2		18.0	18.1		20.1	20.0		23.0	22.4	22.5
Average Penetration	16.3			18.05			20.05			22.63		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	15.09			16.00			13.71			14.29		
Mass of dry soil + container (g)	10.65			11.60			9.61			9.95		
Mass of container (g)	3.23			4.43			3.21			3.40		
Mass of moisture (g)	4.44			4.40			4.10			4.34		
Mass of dry soil (g)	7.42			7.17			6.40			6.55		
Moisture content (%)	59.84			61.37			64.06			66.26		
Liquid Limit (%)	63.66											

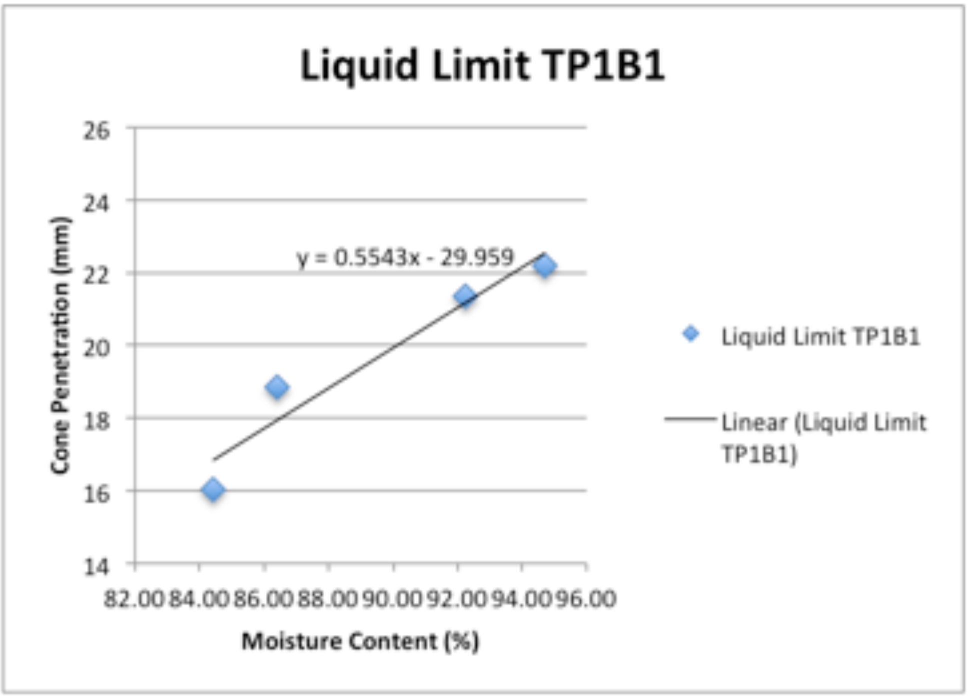
TP2 B2												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	15.9	15.7		18.9	19.8	19.4	22.3	22.8		23.4	23.8	
Average Penetration	15.8			19.4			22.55			23.60		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	12.99			13.00			14.09			14.30		
Mass of dry soil + container (g)	8.76			9.15			9.10			9.23		
Mass of container (g)	3.23			4.43			3.21			3.39		
Mass of moisture (g)	4.23			3.85			4.99			5.07		
Mass of dry soil (g)	5.53			4.72			5.89			5.84		
Moisture content (%)	76.49			81.57			84.72			86.82		
Liquid Limit	81.98											

TP2 B3												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	17.2	16.8		18.6	19.0		21.2	21.4		22.9	22.2	22.0
Average Penetration	17			18.8			21.30			22.37		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	13.49			13.90			14.37			14.77		
Mass of dry soil + container (g)	8.86			9.51			9.08			9.33		
Mass of container (g)	3.23			4.43			3.21			3.39		
Mass of moisture (g)	4.63			4.39			5.29			5.44		
Mass of dry soil (g)	5.63			5.08			5.87			5.94		
Moisture content (%)	82.24			86.42			90.12			91.58		
Liquid Limit (%)	87.82											

TP3 B1												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	12.4	11.5	12.0	16.0	15.5		23.3	23.1		24.5	24.4	
Average Penetration	11.95			15.75			23.20			24.45		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	9.44			11.49			12.18			10.90		
Mass of dry soil + container (g)	7.09			8.29			8.08			7.25		
Mass of container (g)	3.72			4.10			3.26			3.25		
Mass of moisture (g)	2.35			3.20			4.10			3.65		
Mass of dry soil (g)	3.37			4.19			4.82			4.00		
Moisture content (%)	69.73			76.37			85.06			91.25		
Liquid Limit (%)	82.48											

TP4 B1												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	17.6	17.3		18.3	17.9		21.9	21.7		24.7	24.9	
Average Penetration	17.45			18.1			21.80			24.80		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	13.74			14.08			13.71			14.07		
Mass of dry soil + container (g)	9.58			9.76			9.33			9.44		
Mass of container (g)	3.45			3.53			3.33			3.45		
Mass of moisture (g)	4.16			4.32			4.38			4.63		
Mass of dry soil (g)	6.13			6.23			6.00			5.99		
Moisture content (%)	67.86			69.34			73.00			77.30		
Liquid Limit (%)	71.21											

TP4 B2												
LIQUID LIMIT Test No	1			2			3			4		
Dial gauge reading	15.7	15.5		22.3	22.0		23.5	24.0		25.2	25.5	
Average Penetration	15.6			22.15			23.75			25.35		
Container No	UA1			UA2			UA3			UA4		
Mass of wet soil + container (g)	13.84			13.13			14.42			14.35		
Mass of dry soil + container (g)	9.27			8.68			9.23			9.15		
Mass of container (g)	3.46			3.53			3.33			3.45		
Mass of moisture (g)	4.57			4.45			5.19			5.20		
Mass of dry soil (g)	5.81			5.15			5.90			5.70		
Moisture content (%)	78.66			86.41			87.97			91.23		
Liquid Limit	83.92											


Liquid Limit TP1B1
Cone Penetration (mm)
26
24
22
20
18
16
14
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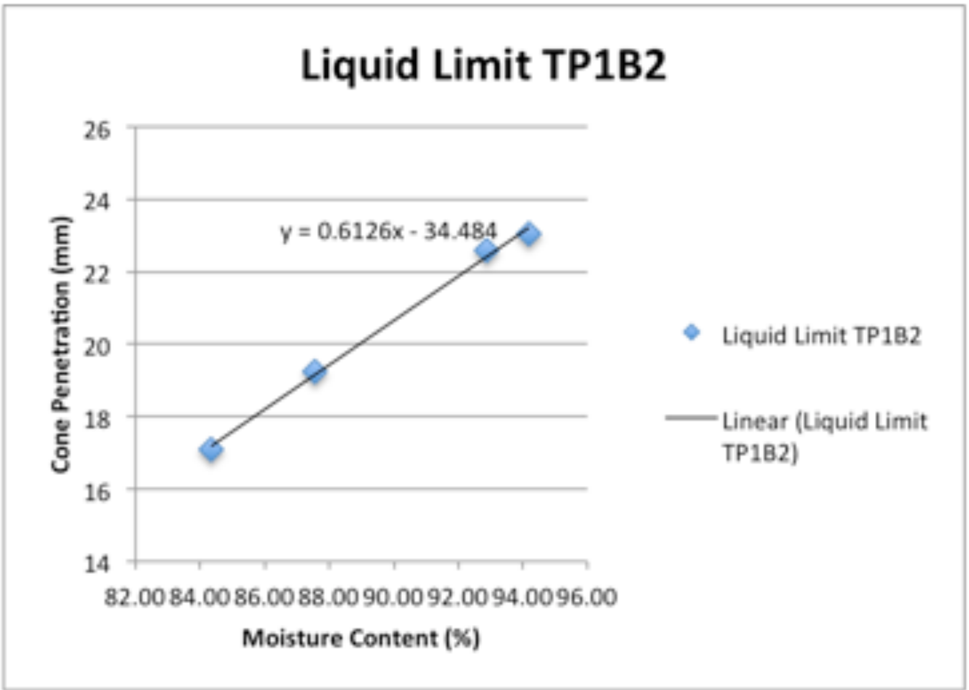
82.00 84.00 86.00 88.00 90.00 92.00 94.00 96.00
Moisture Content (%)
Liquid Limit TP1B1
Linear (Liquid Limit TP1B1)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Cone Penetration (mm)
26
24
22
20
18
16
14
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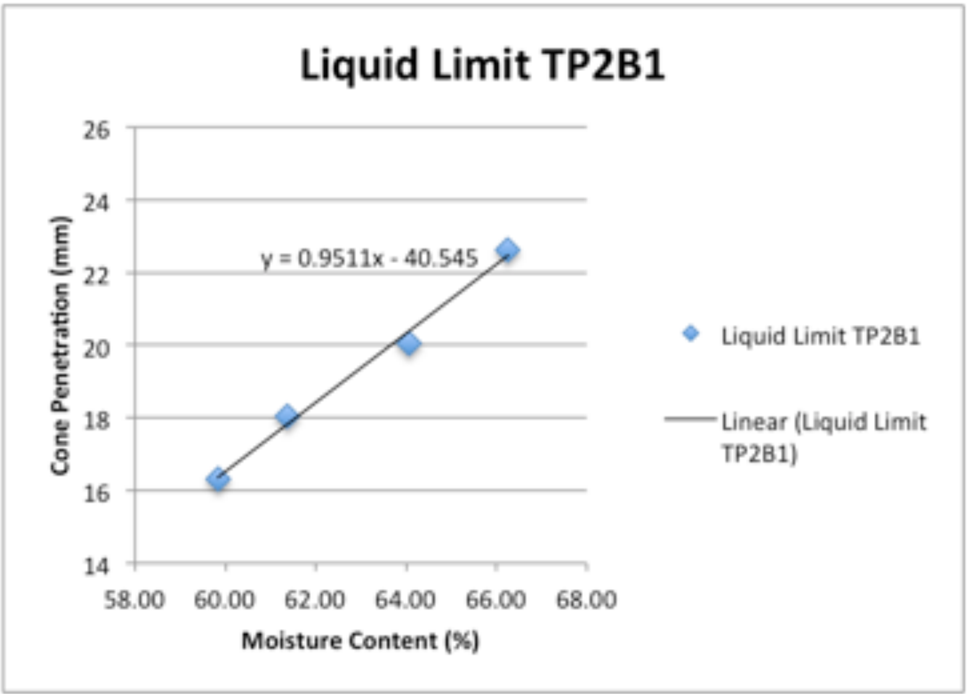
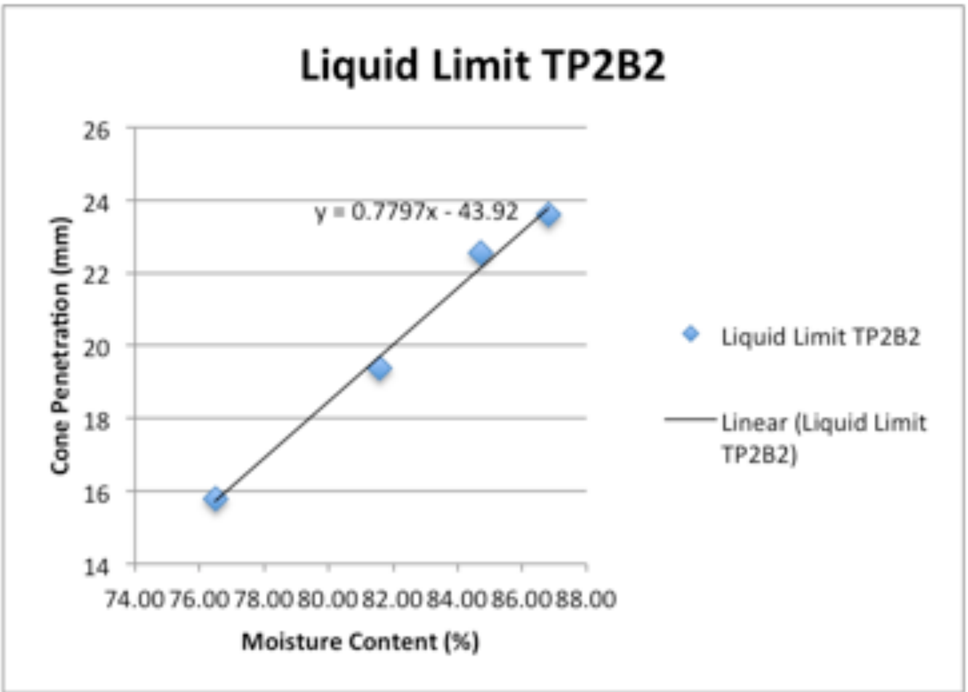
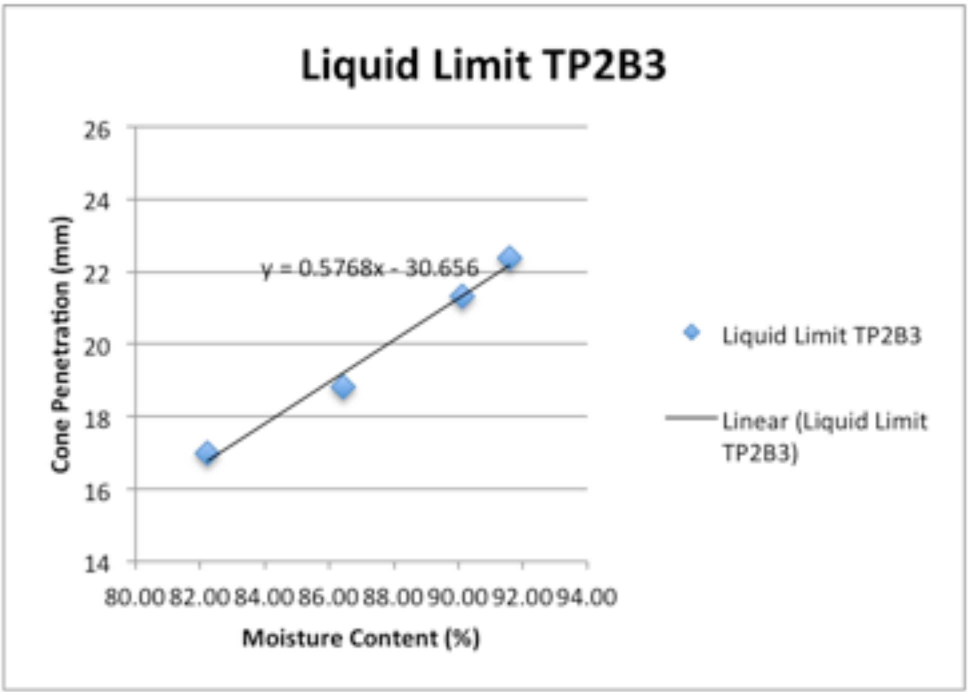
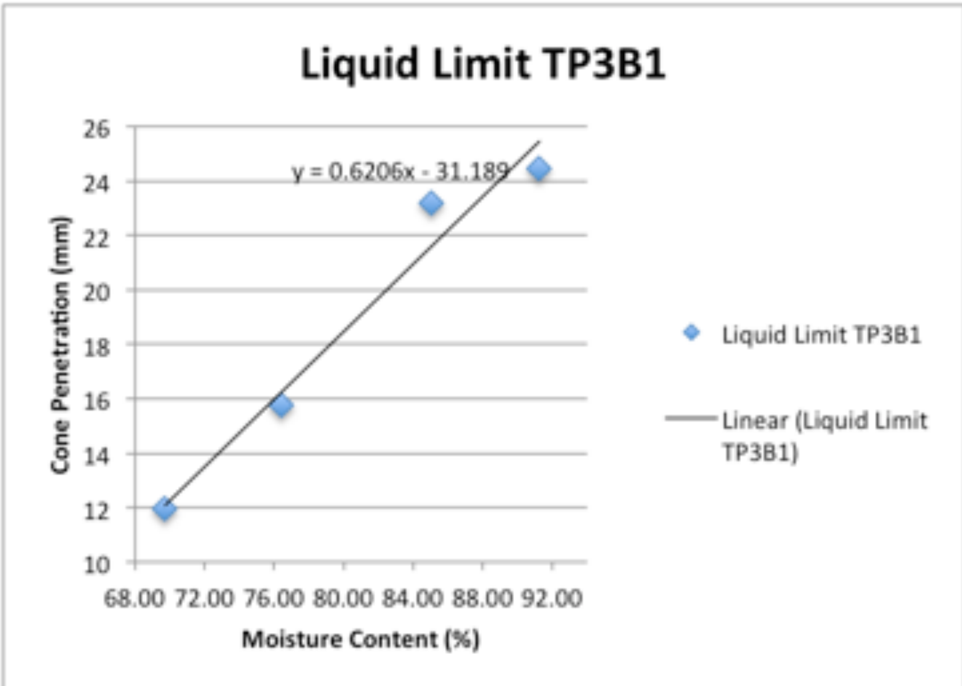
Moisture Content (%)
Liquid Limit TP1B2
Linear (Liquid Limit TP1B2)


Liquid Limit TP2B1
26
24
22
20
18
16
14
Cone Penetration (mm)
y = 0.9511x - 40.545
58.00
60.00
62.00
64.00
66.00
68.00
Moisture Content (%)
Liquid Limit TP2B1
Linear (Liquid Limit TP2B1)


Liquid Limit TP2B2
26
24
22
20
18
16
14
Cone Penetration (mm)
y = 0.7797x - 43.92
74.00
76.00
78.00
80.00
82.00
84.00
86.00
88.00
Moisture Content (%)
Liquid Limit TP2B2
Linear (Liquid Limit TP2B2)


Liquid Limit TP2B3
y = 0.5768x - 30.656
Cone Penetration (mm)
26
24
22
20
18
16
14
80.00 82.00 84.00 86.00 88.00 90.00 92.00 94.00
Moisture Content (%)
Liquid Limit TP2B3
Linear (Liquid Limit TP2B3)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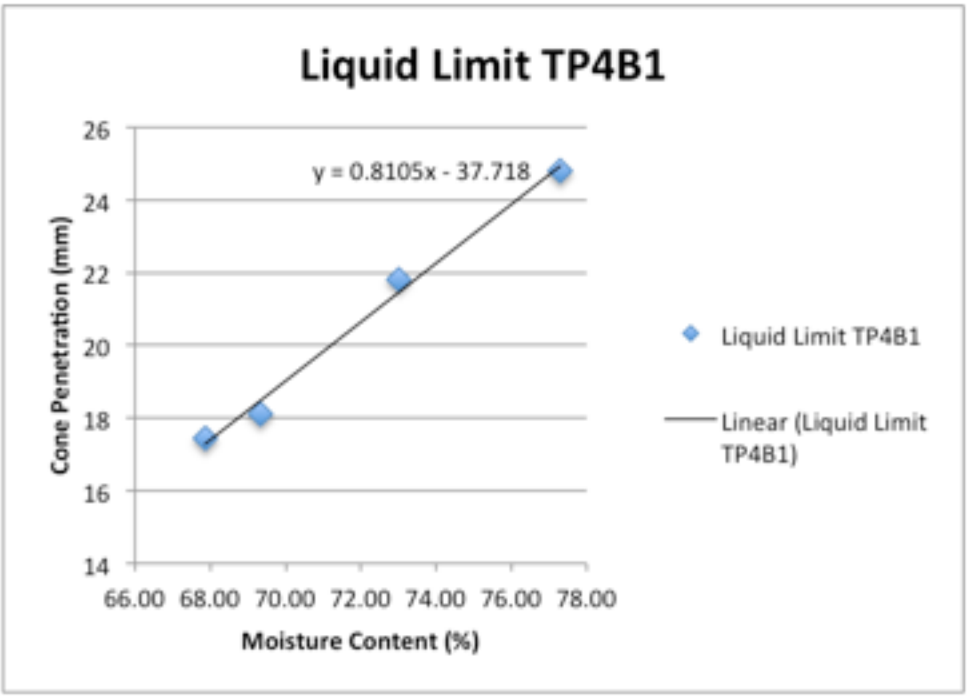
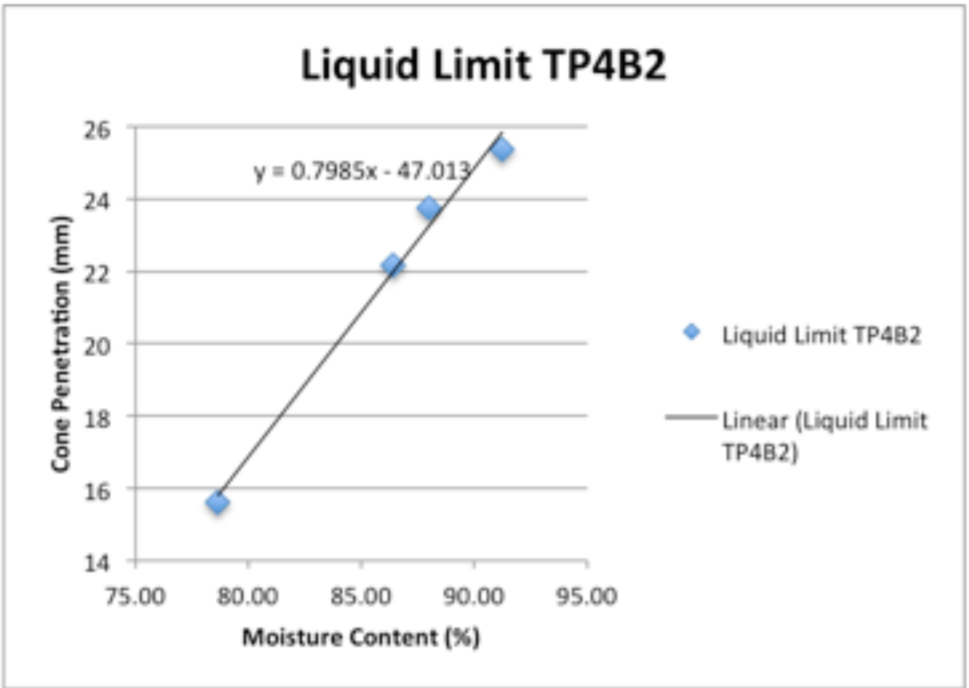
Cone Penetration (mm)
26
24
22
20
18
16
14
12
10
68.00 72.00 76.00 80.00 84.00 88.00 92.00
Moisture Content (%)
Liquid Limit TP3B1
Linear (Liquid Limit TP3B1)


Liquid Limit TP4B1
y = 0.8105x - 37.718
Cone Penetration (mm)
26
24
22
20
18
16
14
66.00 68.00 70.00 72.00 74.00 76.00 78.00
Moisture Content (%)
Liquid Limit TP4B1
Linear (Liquid Limit TP4B1)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y = 0.7985x - 47.013
Cone Penetration (mm)
26
24
22
20
18
16
14
75.00 80.00 85.00 90.00 95.00
Moisture Content (%)
Liquid Limit TP4B2
Linear (Liquid Limit TP4B2)

Plastic Limit Test Results

PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP1B1
Mass of wet soil + container (g)	15.55	15.19	15.09	14.31	
Mass of dry soil + container (g)	12.22	11.85	11.83	10.95	
Mass of container (g)	5.60	5.33	5.30	4.41	
Mass of moisture (g)	3.33	3.34	3.26	3.36	
Mass of dry soil (g)	6.62	6.52	6.53	6.54	
Moisture content (%)	50.30	51.23	49.92	51.38	50.71

PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP1B2
Mass of wet soil + container (g)	15.37	15.36	15.08	13.80	
Mass of dry soil + container (g)	12.13	12.13	12.04	10.82	
Mass of container (g)	5.64	5.39	5.30	4.40	
Mass of moisture (g)	3.24	3.23	3.04	2.98	
Mass of dry soil (g)	6.49	6.74	6.74	6.42	
Moisture content (%)	49.92	47.92	45.10	46.42	47.34

PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP2B1
Mass of wet soil + container (g)	15.71	15.53	15.57	16.74	
Mass of dry soil + container (g)	12.83	12.74	12.86	13.49	
Mass of container (g)	5.52	5.38	5.51	4.82	
Mass of moisture (g)	2.88	2.79	2.71	3.25	
Mass of dry soil (g)	7.31	7.36	7.35	8.67	
Moisture content (%)	39.40	37.91	36.87	37.49	37.92

PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP2B2
Mass of wet soil + container (g)	14.56	15.02	14.89	15.18	
Mass of dry soil + container (g)	11.63	11.83	11.74	11.82	
Mass of container (g)	5.52	5.37	5.51	4.80	
Mass of moisture (g)	2.93	3.19	3.15	3.36	
Mass of dry soil (g)	6.11	6.46	6.23	7.02	
Moisture content (%)	47.95	49.38	50.56	47.86	48.94

PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP2B3
Mass of wet soil + container (g)	15.60	14.79	15.01	13.19	
Mass of dry soil + container (g)	12.36	11.79	11.88	10.50	
Mass of container (g)	5.52	5.39	5.51	4.80	
Mass of moisture (g)	3.24	3.00	3.13	2.69	
Mass of dry soil (g)	6.84	6.40	6.37	5.70	
Moisture content (%)	47.37	46.88	49.14	47.19	47.64

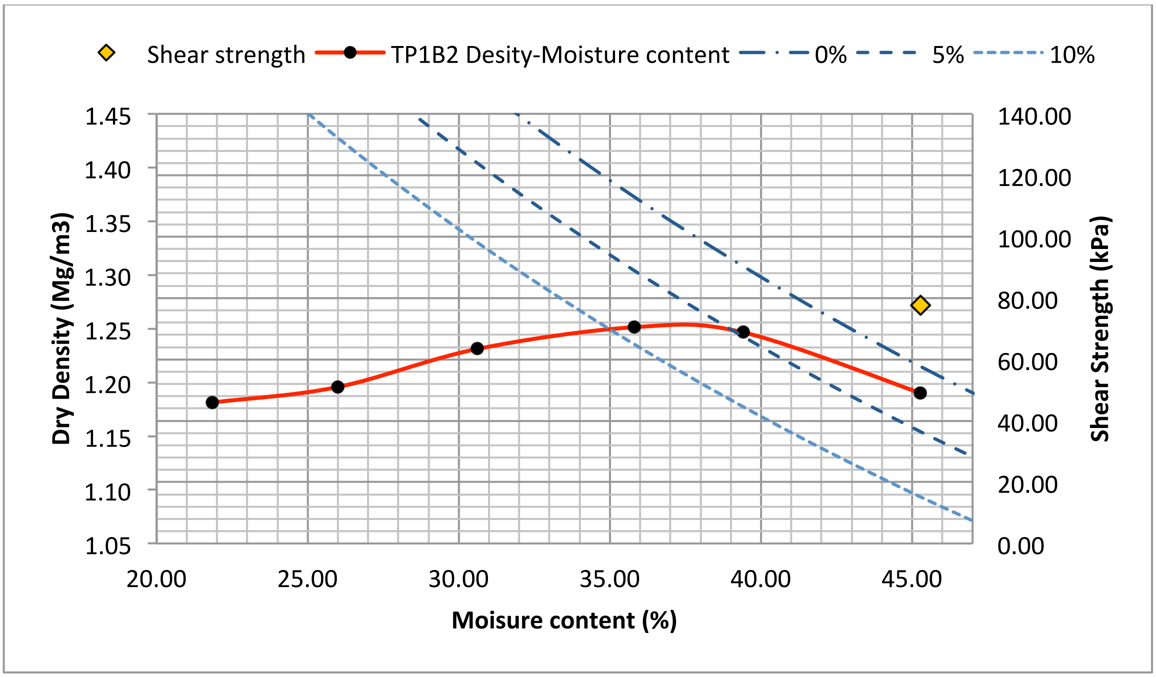
PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP3B1
Mass of wet soil + container (g)	32.21	16.07	15.64	15.02	
Mass of dry soil + container (g)	28.86	12.65	12.55	11.79	
Mass of container (g)	21.00	5.34	5.31	4.41	
Mass of moisture (g)	3.35	3.42	3.09	3.23	
Mass of dry soil (g)	7.86	7.31	7.24	7.38	
Moisture content (%)	42.62	46.79	42.68	43.77	43.96

PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP4B1
Mass of wet soil + container (g)	14.92	14.15	14.46	12.39	
Mass of dry soil + container (g)	12.13	11.46	11.70	10.04	
Mass of container (g)	5.47	4.75	5.43	4.65	
Mass of moisture (g)	2.79	2.69	2.76	2.35	
Mass of dry soil (g)	6.66	6.71	6.27	5.39	
Moisture content (%)	41.89	40.09	44.02	43.60	42.40

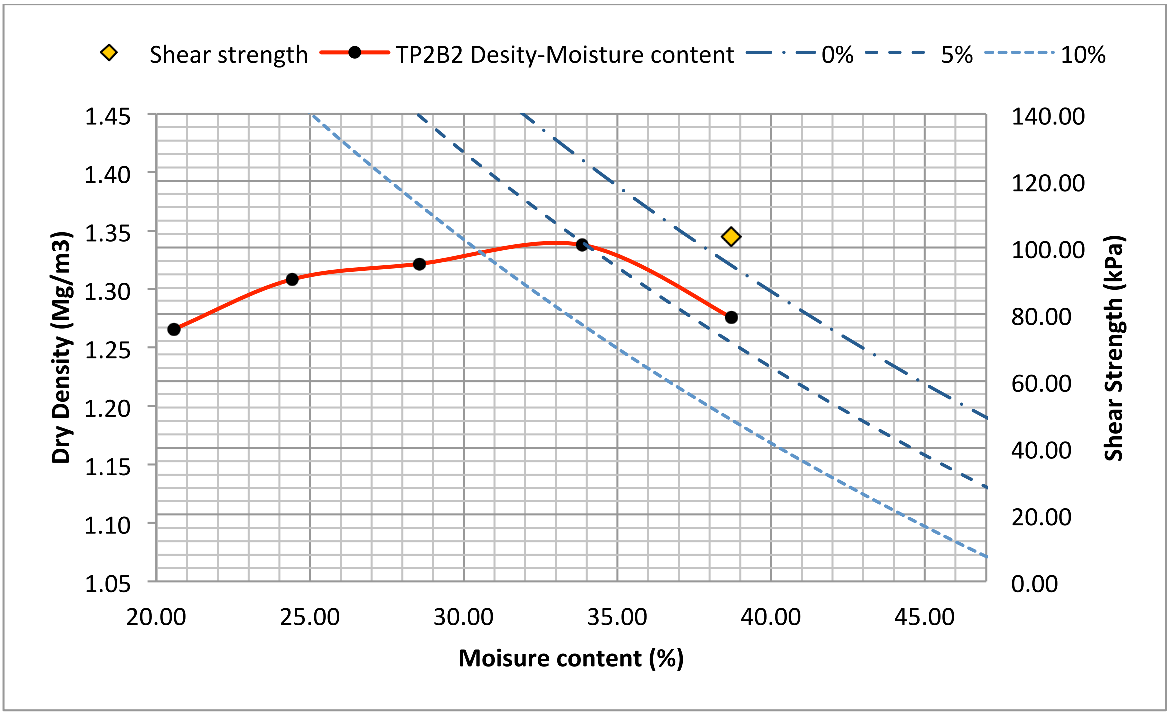
PLASTIC LIMIT Test No	1	2	3	4	Average
Container No	UA PL1	UA PL2	UA PL3	UA PL4	Sample No: TP4B2
Mass of wet soil + container (g)	14.08	14.84	14.25	12.23	
Mass of dry soil + container (g)	11.09	11.28	11.07	9.56	
Mass of container (g)	5.47	4.75	5.43	4.65	
Mass of moisture (g)	2.99	3.56	3.18	2.67	
Mass of dry soil (g)	5.62	6.53	5.64	4.91	
Moisture content (%)	53.20	54.52	56.38	54.38	54.62

Compaction Test Results

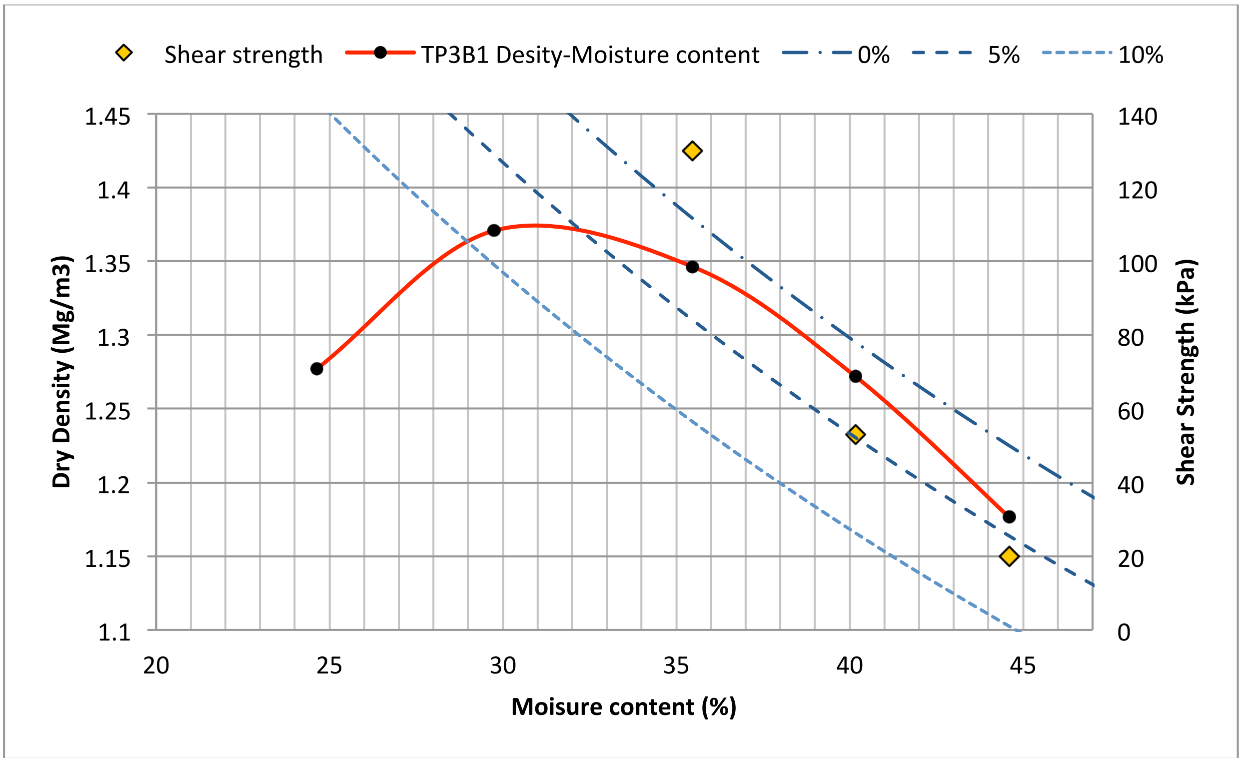
Soil description	Mudstone		**Sample Number**	TP1B2		
Test Method	BS 1377-4:1990 3.3/3.4/3.5/3.6		**Date**	**05.08.2013**		
Procedure	2.5 kg hand rammer					
	3 layers, 27 blow per layer					
	Volume of mould (mm3): 993.89					
Retained on 20mm sieve	0%		**Particle density of soil**			**2.7**
Test No.	**1**	**2**	**3**	**4**	**5**	**6**
Mass of mould+base and extension (m1) gr	4974.7	4974.7	4974.7	4974.7	4974.7	4974.7
Mass of mould+base and extension+compacted specimen (m2) gr	6405	6472	6573	6664	6702	6693
Mass of compacted specimen (m2-m1) gr	1430.3	1497.3	1598.3	1689.3	1727.3	1718.3
Bulk density (ρ=[m2-m1]/v) Mg/m3	1.44	1.51	1.61	1.70	1.74	1.73
Moisture content (w) %	21.85	25.99	30.62	35.82	39.42	45.28
Dry density (ρd=100ρ/[100+w])	1.18	1.20	1.23	1.25	1.25	1.19
Shear strength (kPa)	>150	>150	>150	>150	>150	77.67
Maximum dry density (Mg/m3)	1.25					
Optimum moisture content (%)	37.4					



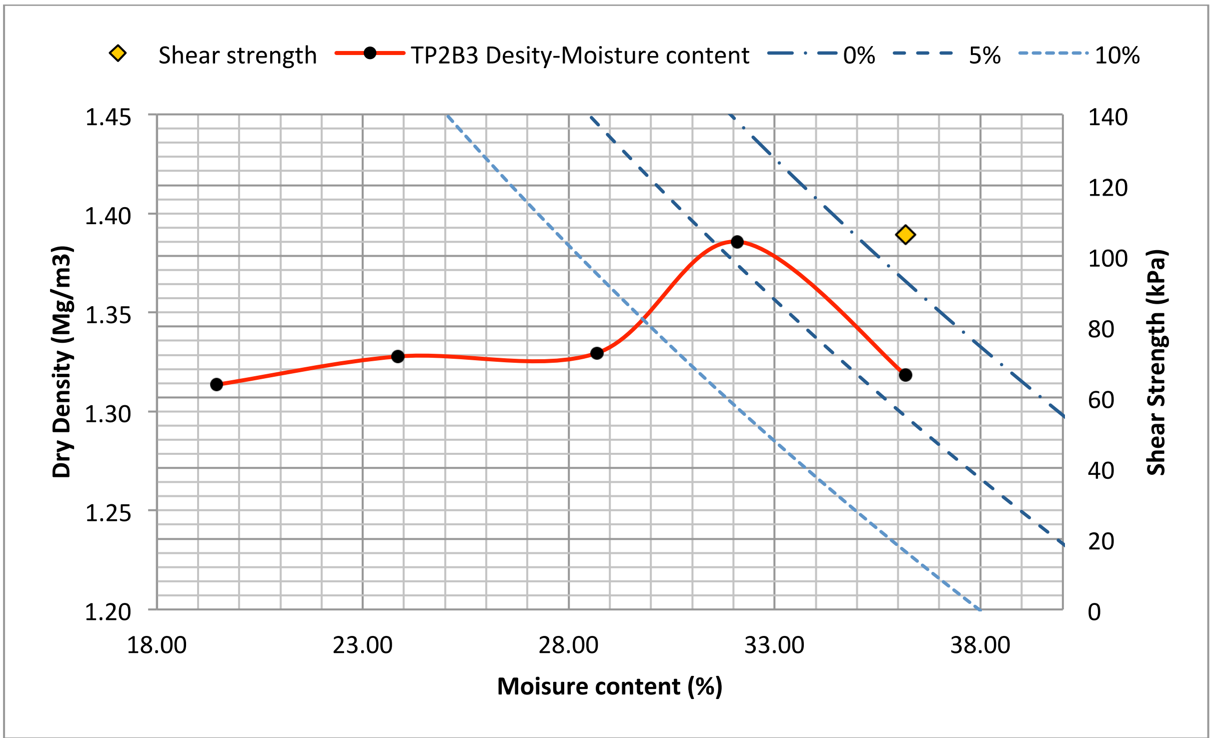
Soil description	Mudstone		**Sample Number**	TP2B2	
Test Method	BS 1377-4:1990: 3.3/3.4/3.5/3.6		**Date**	**05.08.2013**	
Procedure	2.5 kg hand rammer				
	3 layers, 27 blow per layer				
	Volume of mould (mm3): 993.89				
Retained on 20mm sieve	0%		**Particle density of soil**		**2.7**
Test No.	**1**	**2**	**3**	**4**	**5**
Mass of mould+base and extension (m1) gr	4976.1	4976.1	4976.1	4976.1	4976.1
Mass of mould+base and extension+compacted specimen (m2) gr	6493	6594	6665	6756	6735
Mass of compacted specimen (m2-m1) gr	1516.9	1617.9	1688.9	1779.9	1758.9
Bulk density (ρ=[m2-m1]/v) Mg/m3	1.53	1.63	1.70	1.79	1.77
Moisture content (w) %	20.58	24.42	28.57	33.86	38.70
Dry density (ρd=100ρ/[100+w])	1.27	1.31	1.32	1.34	1.28
Shear strength (kPa)	>150	>150	>150	>150	103.33
Maximum dry density (Mg/m3)	1.34				
Optimum moisture content (%)	32.8				



Soil description	Mudstone		**Sample Number**	TP3B1	
Test Method	BS 1377-4:1990: 3.3/3.4/3.5/3.6		**Date**	**02.08.2013**	
Procedure	2.5 kg hand rammer 3 layers, 27 blow per layer Volume of mould (cm^3): 993.89				
Retained on 20mm sieve	0%		**Particle density of soil**		**2.7**
Test No.	**1**	**2**	**3**	**4**	**5**
Mass of mould+base and extension (m1) gr	4975.2	4975.2	4975.2	4975.2	4975.2
Mass of mould+base and extension+compacted specimen (m2) gr	6557	6743	6788	6747	6666
Mass of compacted specimen (m2-m1) gr	1581.8	1767.8	1812.8	1771.8	1690.8
Bulk density (ρ=[m2-m1]/v) Mg/m3	1.59	1.78	1.82	1.78	1.70
Moisture content (w) %	24.63	29.75	35.48	40.17	44.59
Dry density (ρd=100ρ/[100+w])	1.28	1.37	1.35	1.27	1.18
Shear strength (kPa)	>150	>150	130	53	20
Maximum dry density (Mg/m3)	1.37				
Optimum moisture content (%)	31.2				



Soil description	Mudstone		**Sample Number**	TP2B3	
Test Method	BS 1377-4:1990: 3.3/3.4/3.5/3.6		**Date**	**06.08.2013**	
Procedure	2.5 kg hand rammer 3 layers, 27 blow per layer Volume of mould (mm3): 993.89				
Retained on 20mm sieve	0%		**Particle density of soil**		**2.7**
Test No.	**1**	**2**	**3**	**4**	**5**
Mass of mould+base and extension (m1) gr	4974.6	4974.6	4974.6	4974.6	4974.6
Mass of mould+base and extension+compacted specimen (m2) gr	6534	6609	6675	6794	6759
Mass of compacted specimen (m2-m1) gr	1559.4	1634.4	1700.4	1819.4	1784.4
Bulk density (ρ=[m2-m1]/v) Mg/m3	1.57	1.64	1.71	1.83	1.80
Moisture content (w) %	19.45	23.85	28.68	32.10	36.19
Dry density (ρd=100ρ/[100+w])	1.31	1.33	1.33	1.39	1.32
Shear strength (kPa)	>150	>150	>150	>150	106
Maximum dry density (Mg/m3)	1.38				
Optimum moisture content (5)	32.8				



Shear Vane Results

Vane shear test results (hand aparatus)						
TP1B2	Test 1	Test 2	Test 3	Test 4	Test 5	Test 6
Dry density (Mg/m3)	1.18	1.20	1.23	1.25	1.25	1.19
Mositure content (%)	21.85	25.99	30.62	35.82	39.42	45.28
Shear strength at centre (kPa)	>150	>150	>150	>150	>150	80.00
Shear strength at point 2 (kPa)	>150	>150	>150	>150	>150	72.00
Shear strength at point 3 (kPa)	>150	>150	>150	>150	>150	81.00
Average shear strength	>150	>150	>150	>150	>150	77.67

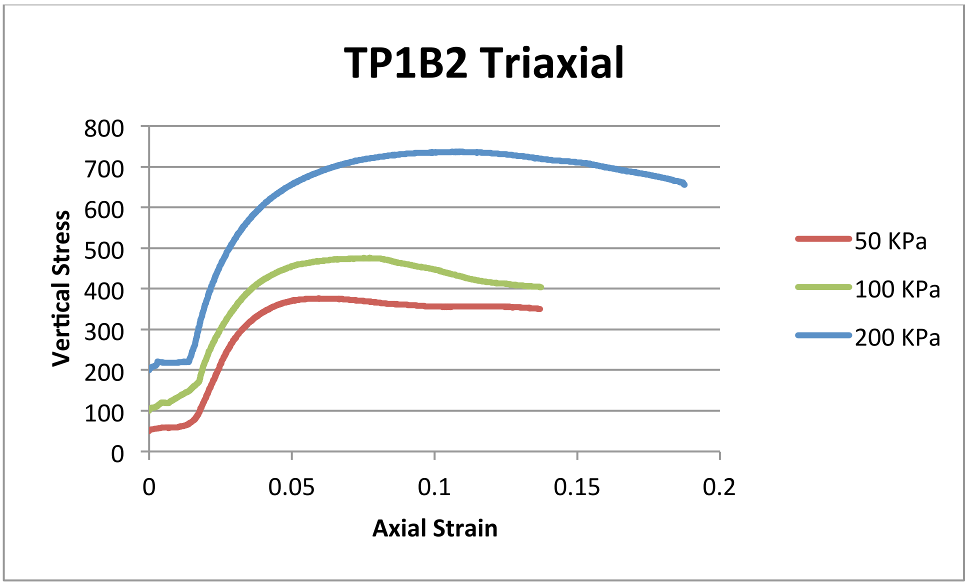
Vane shear test results (hand aparatus)					
TP2B2	Test 1	Test 2	Test 3	Test 4	Test 5
Dry density (Mg/m3)	1.27	1.31	1.32	1.34	1.28
Mositure content (%)	20.58	24.42	28.57	33.86	38.70
Shear strength at centre (kPa)	>150	>150	>150	>150	105
Shear strength at point 2 (kPa)	>150	>150	>150	>150	86
Shear strength at point 3 (kPa)	>150	>150	>150	>150	119
Average shear strength	>150	>150	>150	>150	103.3333

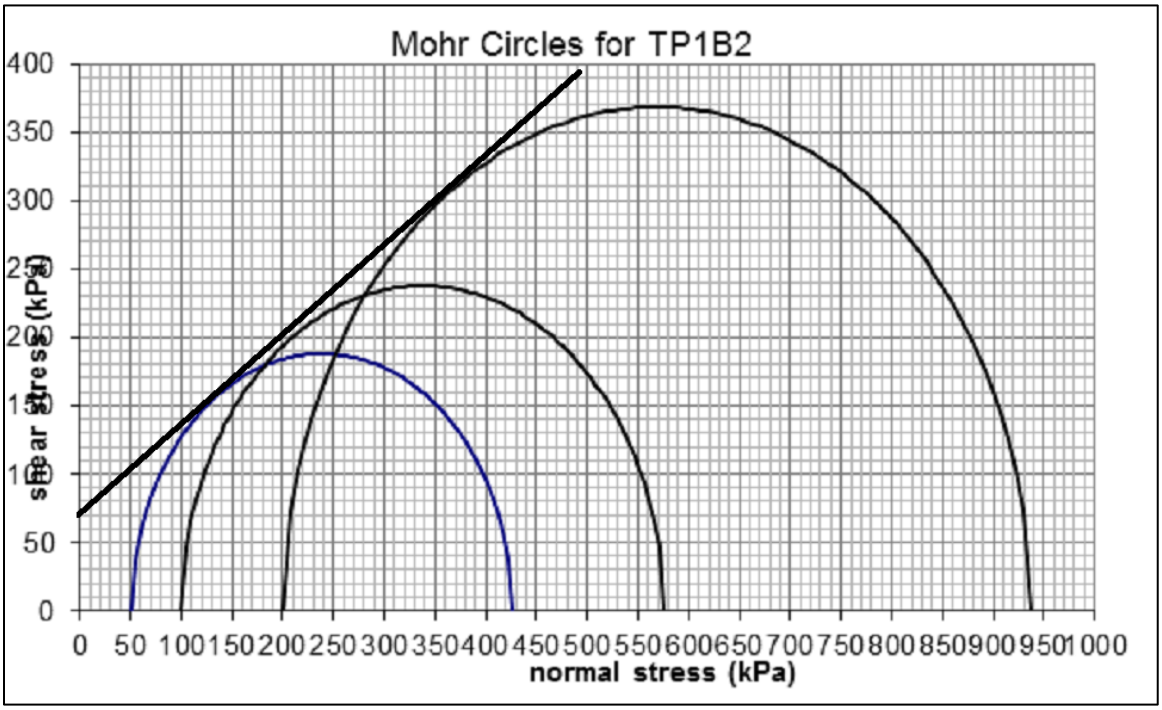
Vane shear test results (hand aparatus)					
TP2B3	Test 1	Test 2	Test 3	Test 4	Test 5
Dry density (Mg/m3)	1.31	1.33	1.33	1.39	1.32
Mositure content (%)	19.45	23.85	28.68	32.10	36.19
Shear strength at centre (kPa)	>150	>150	>150	>150	112
Shear strength at point 2 (kPa)	>150	>150	>150	>150	85
Shear strength at point 3 (kPa)	>150	>150	>150	>150	121
Average shear strength	>150	>150	>150	>150	106

Vane shear test results (hand aparatus)					
TP3B1	Test 1	Test 2	Test 3	Test 4	Test 5
Dry density (Mg/m3)	1.28	1.37	1.35	1.27	1.18
Mositure content (%)	24.63	29.75	35.48	40.17	44.59
Shear strength at centre (kPa)	>150	>150	120	50	20
Shear strength at point 2 (kPa)	>150	>150	120	44	20
Shear strength at point 3 (kPa)	>150	>150	>150	65	20
Average shear strength	>150	>150	130	53	20

Triaxial Test Results

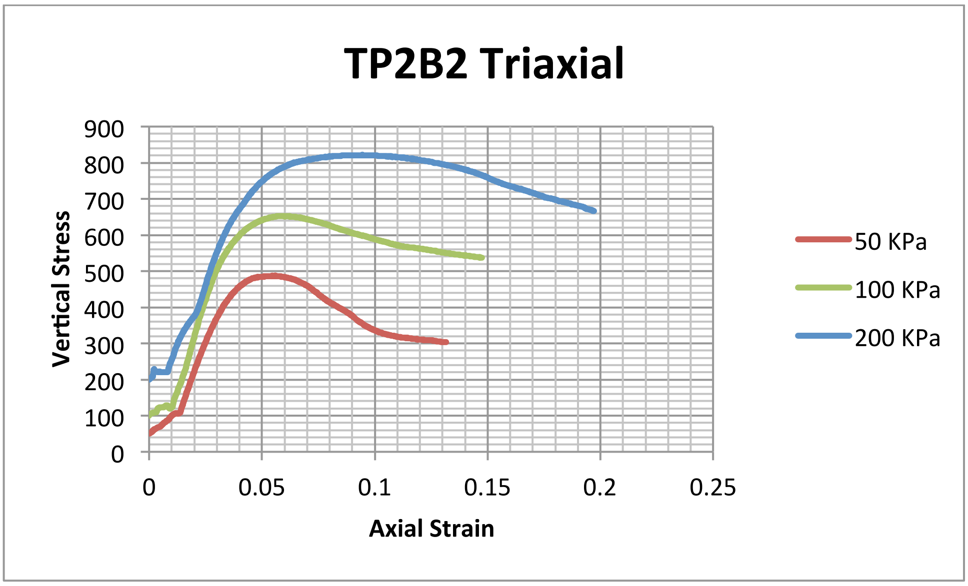
Moisture content determined on the sample at faliure					
Container No:			TP1B2-1	TP2B1-2	TP2B1-3
Wet soil + container (g)			52.63	53.29	61.46
Dry soil + container (g)			42.21	42.59	50.57
Mass of container (g)			14.72	14.04	22.28
	Water content (%)		37.90469	37.47811	38.49417
AVERAGE MOISTURE CONTENT			**37.96**		

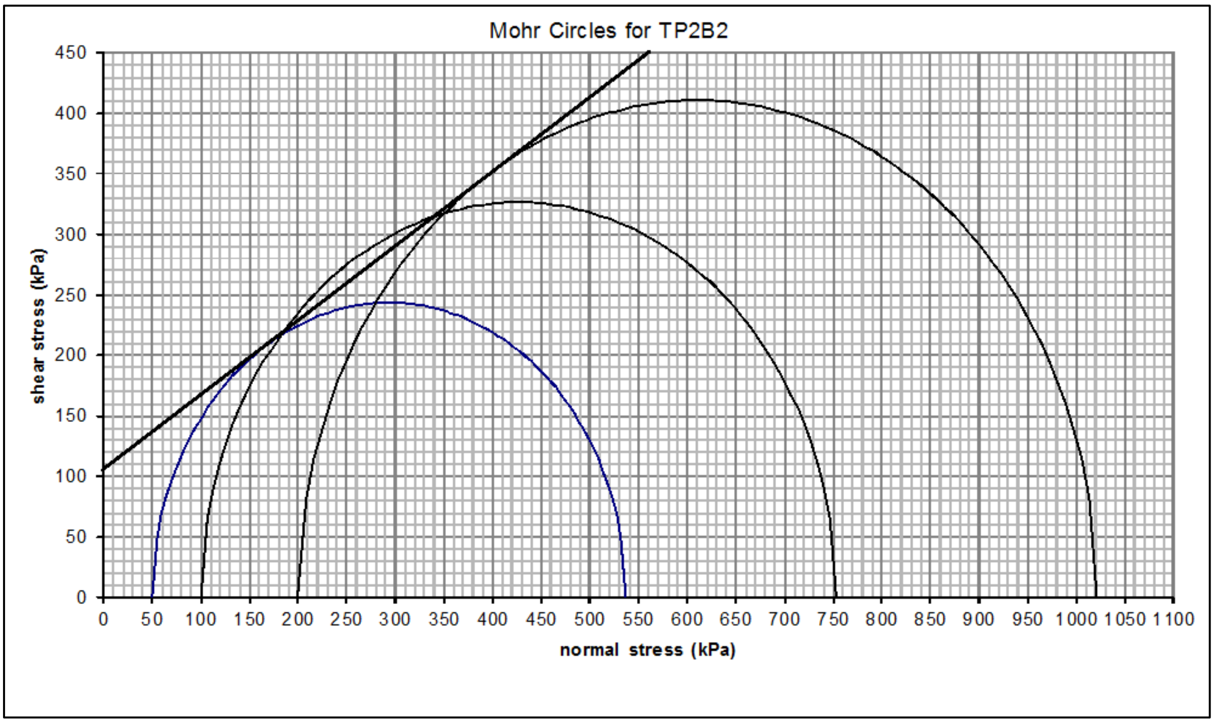




TP1B2 C=70 kpa ϕ= 41°

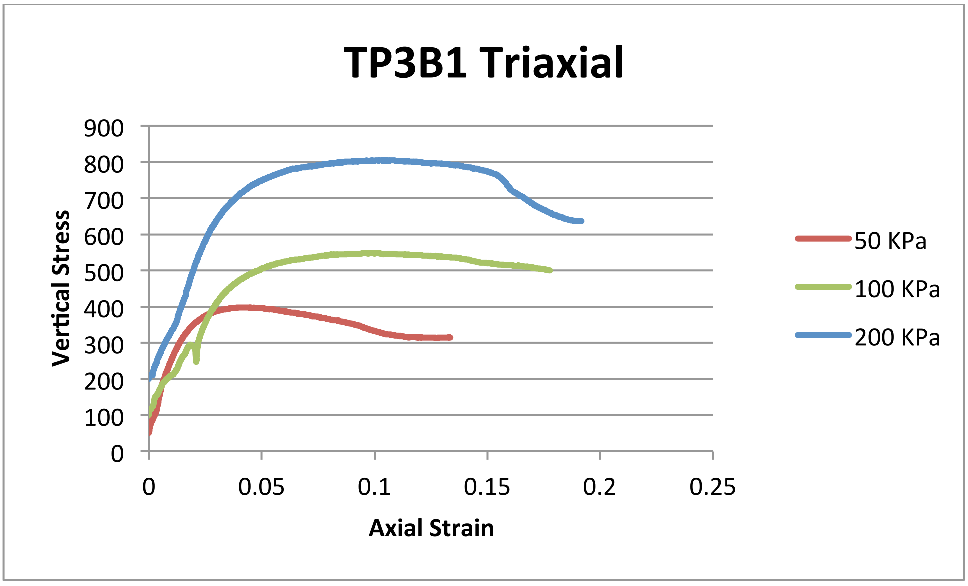
Moisture content determined on the sample at failure (TP2B2)			
Container No:	TP2B2-1	TP2B2-2	TP2B2-3
Wet soil + container (g)	51.9	51.09	59.3
Dry soil + container (g)	42.84	42.53	50.41
Mass of container (g)	14.72	14.04	22.28
Water content (%)	32.21906	30.04563	31.60327
AVERAGE MOISTURE CONTENT	**31.29**		

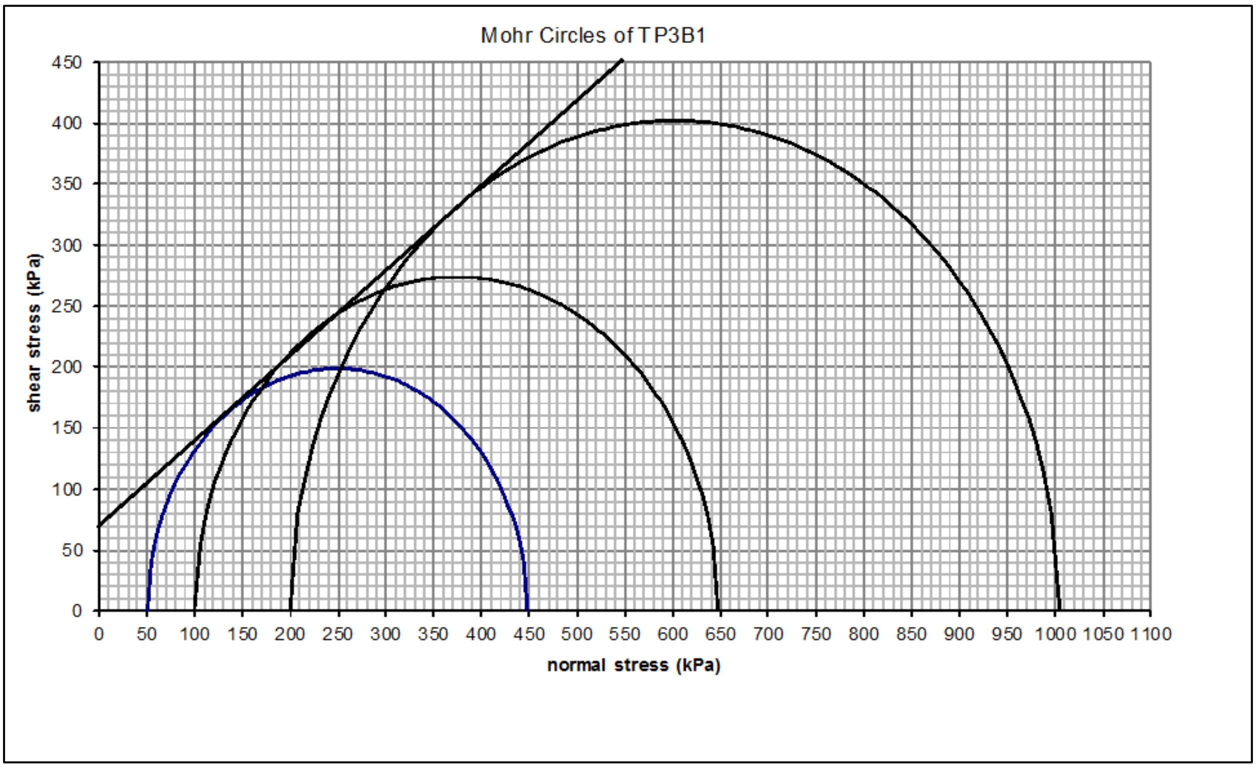




TP2B2 C=107 kpa ϕ= 37°

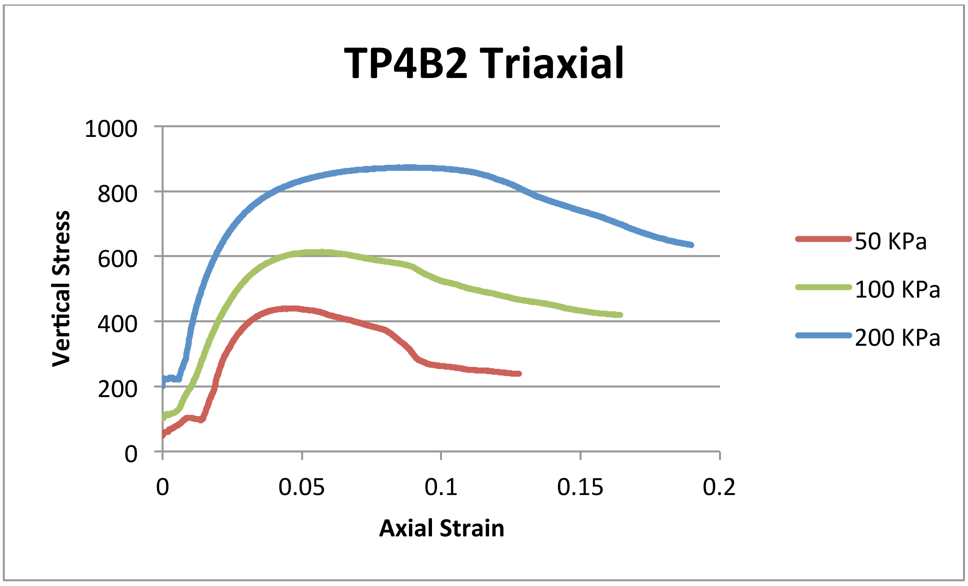
Moisture content determined on the sample at faliure					
Container No:			TP3B1-1	TP3B1-2	TP3B1-3
Wet soil + container (g)			50.88	52.15	56.87
Dry soil + container (g)			42.67	43.42	49.31
Mass of container (g)			14.72	14.04	22.27
	Water content (%)		29.37388	29.71409	27.95858
AVERAGE MOISTURE CONTENT			29.02		

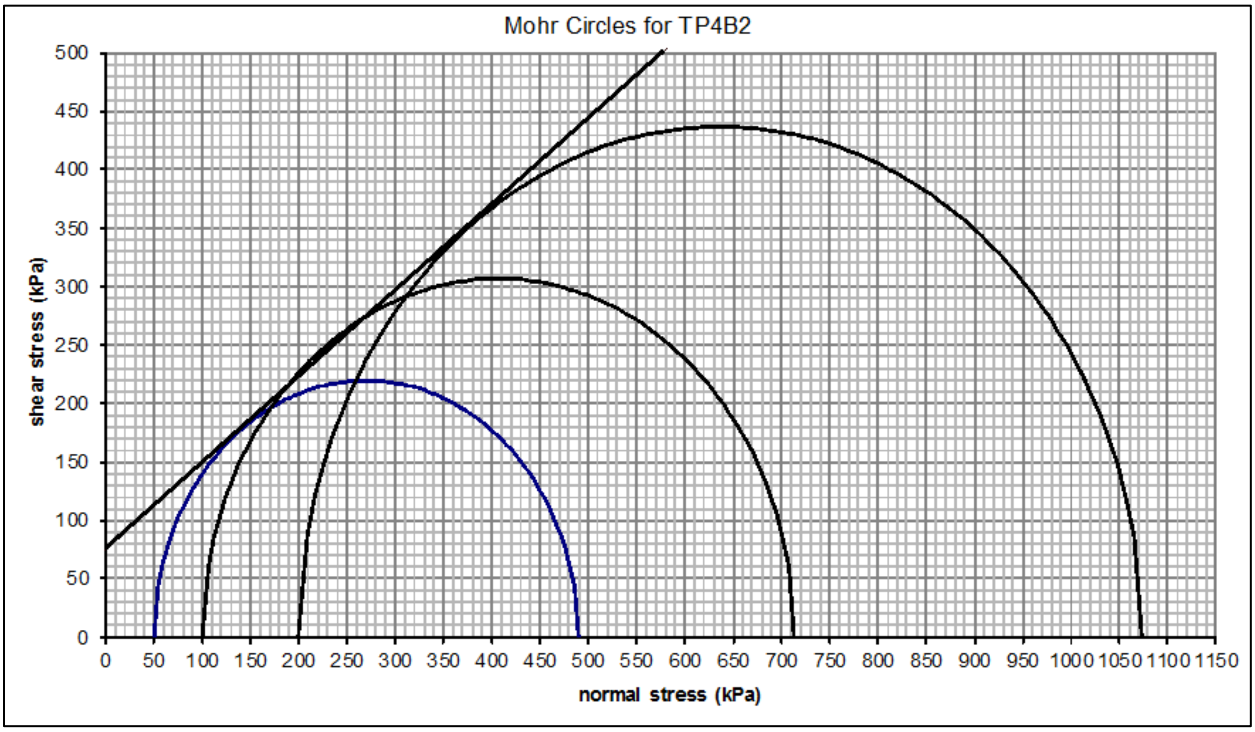




TP3B1 C=60 kpa ϕ= 42°

Moisture content determined on the sample at faliure					
Container No:			TP4B2-1	TP4B2-2	TP4B2-3
Wet soil + container (g)			59.46	54.18	60.41
Dry soil + container (g)			49.68	43.41	50.36
Mass of container (g)			22.96	13.95	22.95
	Water content (%)		36.6018	36.55804	36.66545
AVERAGE MOISTURE CONTENT			**36.61**		





TP4B2 C=64 kpa ϕ= 42°

Appendix B - BGS Borehole Recordings from near the Area

2.

ST 6618 5412 ST65/7 B

RECORD OF WELL (SHAFT OR BORE)

For Survey use only N. 5256

281/166B

Licence No.

EXACT SITE OF WELL

At Midsomer Norton

(No.2 bore)

Town or Village

County Somerset.

Six-inch quarter sheet ~~20SW~~, 29NW/E

For Dent, Allcroft & Co.,

State whether owner, tenant, builder, contractor, consultant, etc.:—

Address (if different from above) Radstock.

Level of ground surface above sea-level (O.D.) +c.305 ft.

If well-top is not at ground level, state how far { above: below; ft.

SHAFT ft.; diameter ft.; Full details of headings (dimensions and directions)

BORE 70 ft.; diameter of bore: at top ins.; at bottom ins.

Full details of permanent lining tubes (position, length, diameter, plain, slotted etc.)

14ft of 8in steel tubes

58ft of 6in perforated tube

Water struck at depths of ft. below well-top.

TEST CONDITIONS

Rest level of water 6ft. above/below well-top. Suction at ft. Yield on hours'/days' test

pumping at 200 galls. per hour with depression to ft. below well-top.

Recovery to rest-level in mins./hours Capacity of pump g.p.h. Date of measurements

NORMAL CONDITIONS

DESCRIPTION OF PERMANENT PUMPING EQUIPMENT:

Make and/or type Motive power

Capacity 200 gallons per hour. Suction at near bottom ft.

Amount pumped galls. per day. Estimated consumption galls. per week.

Well made by F.G.Clements Ltd. Date of well Dec.,1951.

Information from

ADDITIONAL NOTES

ANALYSIS (please attach copy if available)

OD. +c.105.

Inaccessible for W.L. ∵ pump.:

In use.

Visited and sited on Sam. 29NW/E. JR. 25.11.58

Visited site to check ground level.

OD + c. 305. 24/10/67

LOG OF STRATA OVERLEAF.

(1527) Dd574/W137583 12,000 8/54 JC&S Gp659

Geological Survey and Museum, South Kensington, London, S.W.7.		Section 6.	Date Received	1" O.S. Map No.	Site marked on 1" Map	(use symbol) on 6" Map
			14.4.53		⊙	⊙

7

281/166 Messrs. Dent, Allcroft and Co., Ltd., Redfield Road, Midsomer Norton

ST65/7A+B

(a) Surface +315. Bore 80. Lining tubes: 17 x 10 in from surface; 63 x 6 in from 17 down (? perforated). Clements, July 1951.

R.W.L. +c.302 (average). P.W.L. +279. Yield c.460 g.p.h. Electric pump. Nov. 1958.

(b) Surface +305. Lining tubes: 14 x 8 in; 58 x 6 in (perforated). R.W.L. +299. Yield 200 g.p.h. Clements, Dec. 1951.

(b)	KM	...	...	57	57
	CM	...	...	13	70

(For Survey use only) Geological Classification	Nature of Strata (If measurements start below ground surface, state how far ..)	Thickness Feet	Thickness Inches	Depth Feet	Depth Inches
(b) Keuper (Trias) Marl [with Sandstone & Conglomerate beds]	Red Marl	14	-	14	-
	New Red Sandstone	15	6	29	6
	Conglomerate	6	-	35	6
	Soft Sandy Clay	20	6	56	-
	Conglomerate	1	-	57	-
Coal Measures	Coal Measures	13	-	70	-
T.W.J. 2.2.66					

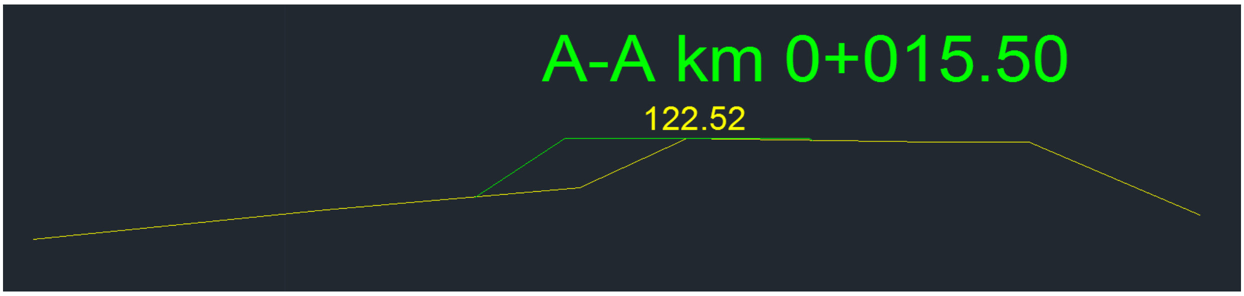
(a) Estimated classification

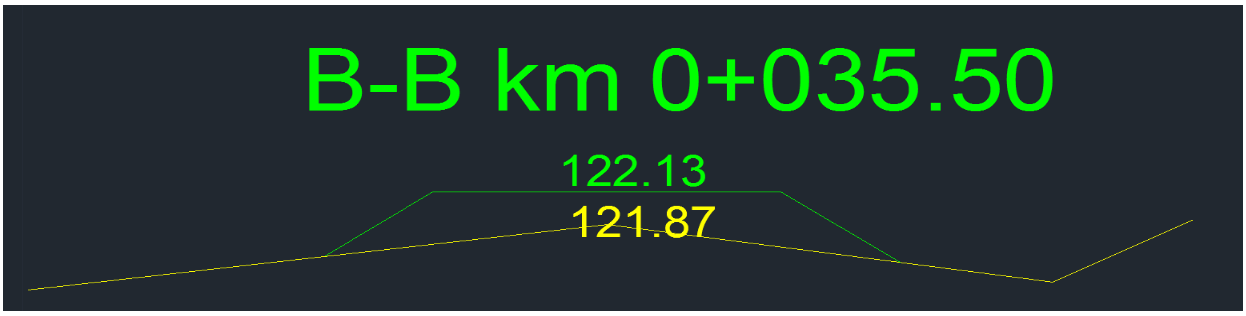
Keuper Marl
Coal Measures } 80 ft 80 ft.

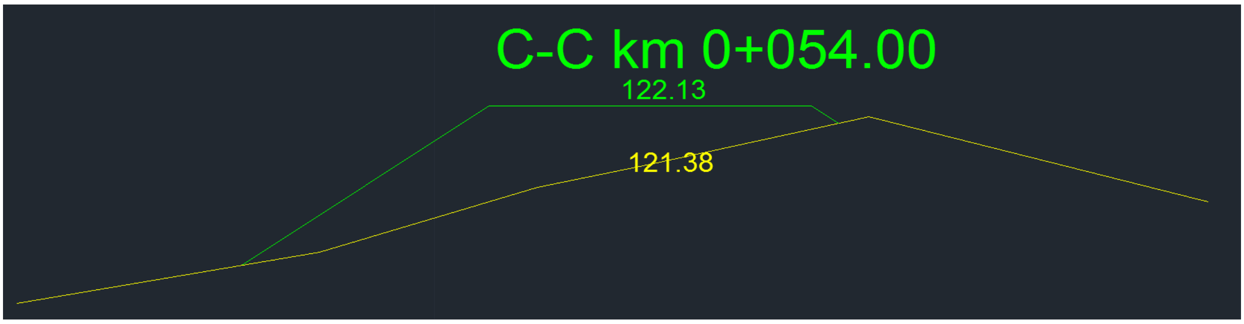
MH.
19/3/73

A: ST 6605 5419
B: ST 6618 5412

Appendix C - Cross-sections of the alignment

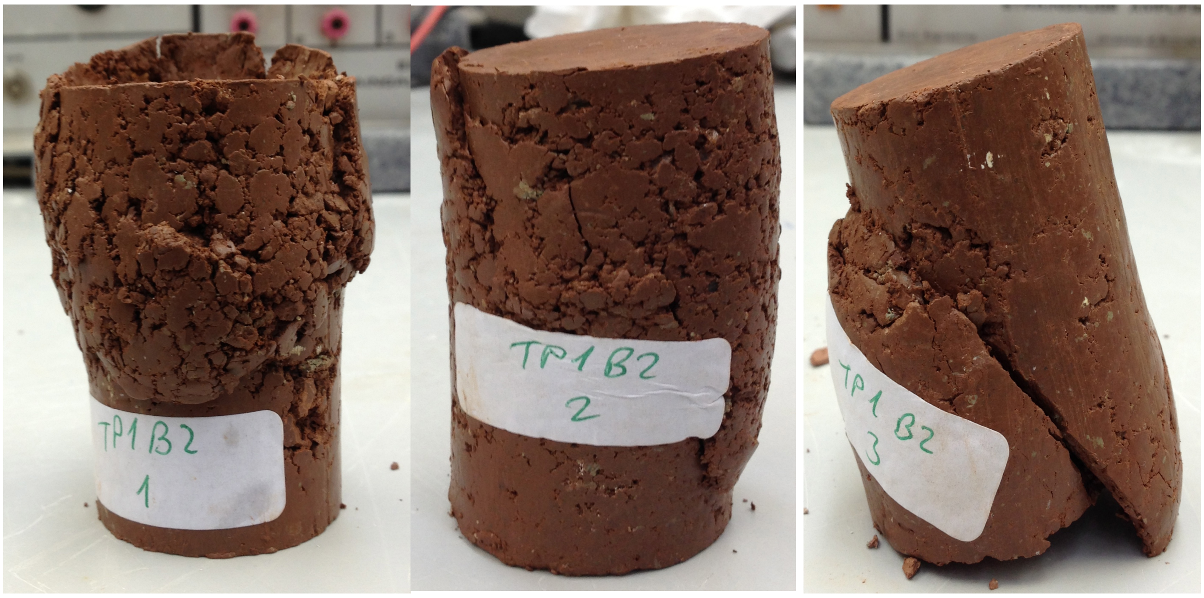




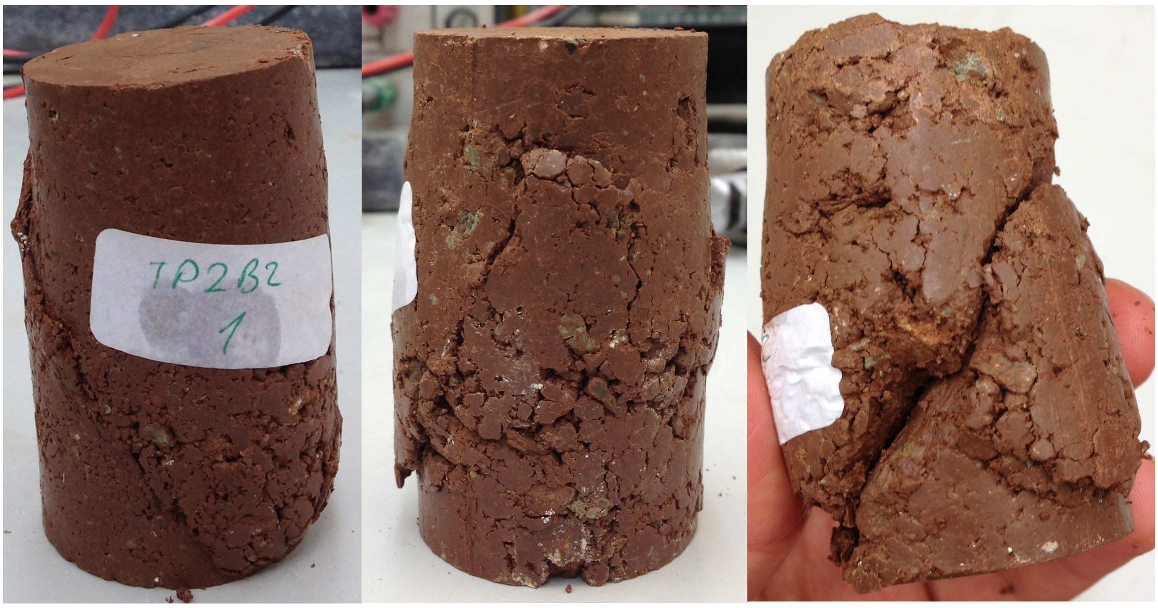


Appendix D - Photos of the triaxial test specimens after shear

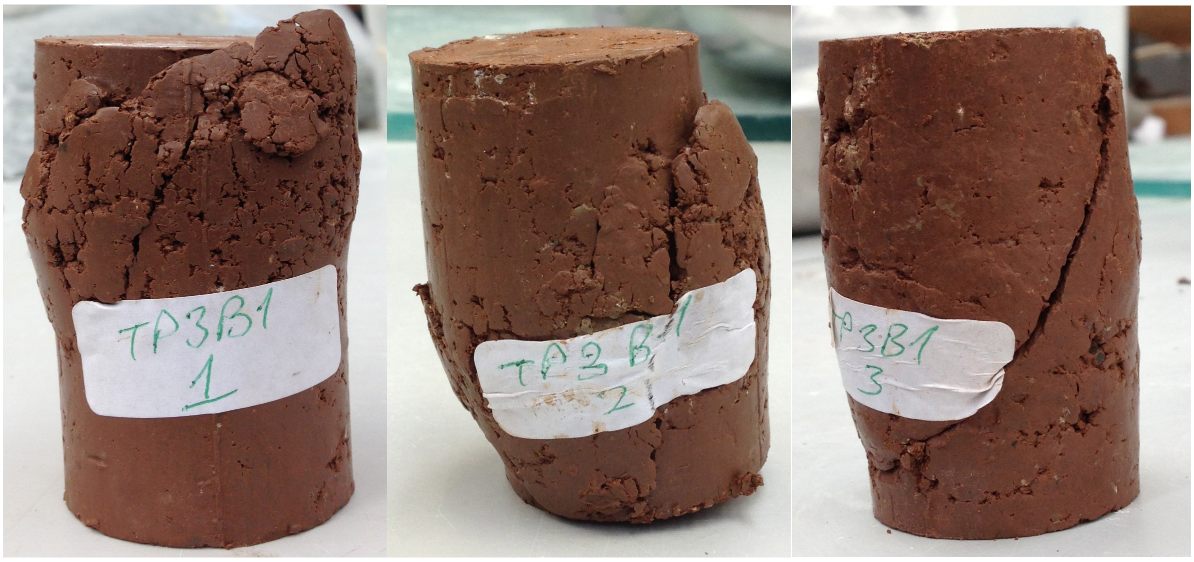
TP1B2



TP2B2



TP3B1



TP4B2