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FACULTY OF ENGINEERING

DEPARTMENT OF CIVIL AND BUILDING ENGINEERING

A comparative analysis of foundations using prescriptive design and static loading test methods

(CASE STUDY: THE KARUMA INTERCONNECTION POWER PROJECT IN UGANDA)

BY

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A dissertation submitted to the Kyambogo University Graduate School in partial fulfilment of the requirements for the award of Masters of Science degree in Structural Engineering.

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CERTIFICATION

The undersigned certify that they have read and hereby recommend for acceptance by Kyambogo University, a dissertation entitled: **“A comparative analysis of foundations using prescriptive design and static loading test methods”**, by **Acidri Samuel**, in partial fulfilment of the requirements for the award of a degree of **Master of Science in Structural Engineering** of Kyambogo University.

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DECLARATION

I, **Acidri Samuel**, hereby declare that this submission is my own work and that, to the best of my knowledge and belief, it contains no material previously published or written by another person nor material which has been accepted for the award of any other degree of the university or other institute of higher learning, except where due acknowledgement has been made in the text and reference list.

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LIST OF ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
BS	British Standard code(s) of practice
CGS	Canadian Geotechnical Society
CH	Clays of High Plasticity
CI	Clays of Intermediate Plasticity
CL	Clays of Low Plasticity
EPCC	Engineering, Procurement and Construction Contractor
FHWA	The Federal Highway Administration
GB	Giga Byte
GC	Clayey Gravels
GM	Silty Gravels
GOPA	Gesellschaft für Organisation, Planung und Ausbildung mbH
GoU	Government of Uganda
GPS	Geographical Positioning Systems
INTEC	International Energy Consultants
ISBN	International Standard Book Number
KIP	Karuma Interconnection Project
km	Kilometre
KPTL	Kalpataru Power Transmission Limited
kV	Kilo Volts

LL	Liquid Limit
Ltd	Limited
MH	Silts of High Plasticity
Min.	Minimum
ML	Silts of Low Plasticity
MoWT	Ministry of Works and Transport
NMC	Natural Moisture Content
OMC	Optimum Moisture Content
PDCA	Painting and Decorating Contractors of America
PI	Plasticity Index
PL	Plastic Limit
ROW	Right of Way
SBC	Soil Bearing Capacity
SC	Clayey Sands
SI	Système Internationale (International System of Units)
SM	Silty Sands
SPT	Standard Penetration Test
TL	Transmission Line
TWR	Tower
USh / UGx	Uganda Shilling
UTM	Universal Transverse Mercator
%	Per cent
°	Degree angle

ABSTRACT

Foundations for overhead transmission line lattice towers are subjected to overturning loads imposed by winds and impact. Hence, they are designed to resist uplift, compression, lateral and interconnection cable line-tension forces. However, due to the non-linear nature of the load-displacement response for typical transmission line foundations, uncertainties in subsoil behaviour and design models, variations in soil strata, limits of site explorations, and diversity in construction methodologies, the performance of the full scale foundation model was analysed using the reliability-based Static loading test methods.

The researcher used prescriptive design methods in geotechnical investigations, insitu and laboratory tests, geotechnical and structural designs using the load and resistance factor design approach, computer aided design tools, and static load test methods of static axial tensile, static axial compressive, and lateral load tests, to replicate and validate the foundation's insitu long-term sustained load capacity and subsequent displacements.

The findings showed moderately aggressive chemical and environmental conditions, fine-grained soils with low swell potential and medium-dense to hard soil consistencies under 0.3-10m ground water table levels. The static load test results showed a 20% increase in design efficiency, 15-35% cost savings, and 80% reduced displacement overdesigns based on the insitu displacement values of less than 20% of the prescriptive design values.

Although this research has advanced the understanding about transmission line foundation designs using prescriptive design and static loading test methods, further research must be done in quantifying the influence of ambient temperature, weather variations, and rate of insitu backfill-soil compaction on static load test results.

Key words: Foundations, Prescriptive design, Static load test, Transmission Towers

CHAPTER ONE

INTRODUCTION

1.1 Background of the study

Foundation substructures are essential structural members that transmit and distribute different kinds of superstructure loads to the subsoil or rock below without exceeding the bearing capacity of the ground and preventing excessive or uneven settlements, and they are generally classified as shallow or spread and deep foundations (Kaushik *et al*., 2010; Sivakugan and Braja, 2011; Manjriker, 2014). Foundations must fulfil both structural and geotechnical requirements. However, due to uncertainties of the subsoil behaviour, most foundations are either statically or dynamically tested to verify conformity with the design load; the latter is not commonly used in Sub-Saharan Africa since it is a confirmatory test following the failure of a static load test (FHWA-SA-91-042, 1992; Byrne and Berry, 2008).

In Uganda, since 2013, load tests have been used to either prove the maximum capacity of the foundation and/or to verify the predicted design values and settlements under pressure on a foundation that is expendable to the main works; and at times on a working pile whilst limiting the maximum test load to less than 1.5 times the safe working load (Monnet, 2015; Cockerill *et al*., 2017).

Prescriptive design methods such as design codes, calculations and specifications are conservative which leads to overdesigns that stifle innovation and least-cost solutions (Tavares, 2009). Therefore, there is a need to further study and compare the suitability of the prescriptive design methods using the foundation test methods as a design verification tool.

This research, therefore, sought to compare the traditional prescriptive-foundation design method used in Uganda with the newly introduced Static load test method.

1.2 Statement of the Problem

Foundation testing is an important part of the post-foundation construction and/or installation process mainly because of uncertainties in ground conditions and design models (England and Fleming, 1994; Coduto, 2001; Tipter, 2018).

Despite recent improvements in site characterisation, exploration strategies, and construction methodologies to reduce uncertainty and variance in analysis, prescriptive design methods have continued to be used with about 66.7% of electric transmission and overhead line design engineers using the prescriptive method of applying traditional global factor of safety ranging from 2.5 to 4.0 depending on the structure type, foundation type, and design model (Cockerill *et al*., 2017; Kandaris and Davidow, 2018).

Due to the nonlinear nature of the load-displacement response for typical transmission line foundations and the prescriptive design's tendency to give linear design solutions unlike static load tests (Kandaris and Davidow, 2018), the prescriptive design method cannot therefore, be solely relied upon in assessing the foundation's load-displacement performance in either axial or lateral load modes (Rodrigo *et al*., 2008; Sivakugan and Braja, 2011; Ministry of Energy and Mineral Development (MoE & MD), 2013; Ghannoum, 2017). This gap, thus, created a need to verify the prescriptive design calculation outputs for a foundation design on a case by case basis using the static load test method as a fundamental and benchmark form of all insitu long-term sustained loading tests (Byrne and Berry, 2008; Monnet, 2015; Tomlinson and Woodward, 2015) in the determination of the foundation's ultimate load bearing capacities and validation of the foundation design assumptions (England and Fleming, 1994).

This research, therefore, showed how typical correlation factors used in prescriptive designs were verified under the insitu static load tests, and it validated the ability of the foundations to support the proposed design loads at predetermined elevations and particular soil strata.

1.3 Objectives of the Study

1.3.1 Main objective

To conduct a comparative analysis of the foundation's performance using the prescriptive design method and insitu static load tests under sustained axial loading.

1.3.2 Specific objectives

The specific objectives of this research were:

- To establish the parameters for geotechnical and structural designs of foundations,
- To carry out geotechnical and structural designs, drafting and detailing of foundation systems based on the parameters,
- To construct insitu full-scale foundation test model with all the quality assurance and quality control checks done, and
- To test the full-scale foundation test model using the insitu static load test method which is the benchmark and most fundamental form of all foundation load test methods; and then analyse the output with reference to the initial prescriptive structural design calculations.

1.4 Research Questions

The following research questions were used during the study:

- What essential tests and parameters are relevant, prior to carrying out the geotechnical and/or structural designs of the foundations?
- What methods can be used to carry out the foundation's geotechnical and/or structural designs, drafting and detailing works?
- What kind of foundation test models shall be constructed for use?
- What load testing methods can be used on the insitu foundation test models to compare the results with those of the prescriptive methods?

1.5 Research Significance

This research study generally had the following significance:

- This study provided insight into the use of static load tests -the benchmark and most fundamental form of all current foundation load test methods- in analysing foundation capacities in Uganda since this is not yet fully adopted by the local engineering fraternity.
- The research study was of great benefit to students in the Civil and Building and/or Structural Engineering fraternities in and Universities and other institutions of higher learning; helping them to gain an in-depth knowledge of the static load test which is the benchmark and most fundamental form of all current foundation load test methods.
- This research offered people who do not belong to the engineering fraternity with information for shaping future national policies about foundation static loading tests -a method for keeping checks on costly conservative designs.

1.6 Scope of Research

1.6.1 Content Scope

This research mainly focused on establishing the foundation's parameters for geotechnical and structural designs, undertaking prescriptive structural design calculations and comparing these results with those of static load tests -the benchmark of all foundation load test methods- on a number of constructed full-scale foundation test models. The models were a total of four (4) for the entire transmission lines as the EPC Contractor, Sinohydro Corporation Ltd had constraints in resources, right of way and land acquisition payment issues.

1.6.2 Geographical Scope

The case study was the Karuma Interconnection Project (KIP) in Uganda, along 248 km Karuma-Kawanda (400 kV) and 77 km Karuma-Lira (132 kV) Transmission Lines (TL).

1.6.3 Time Scope

The research was carried out within a duration of twelve (12) months, starting in the month of August 2018 until August 2019.

1.7 Conceptual Framework of Research

The conceptual frame of the research project was shown below:

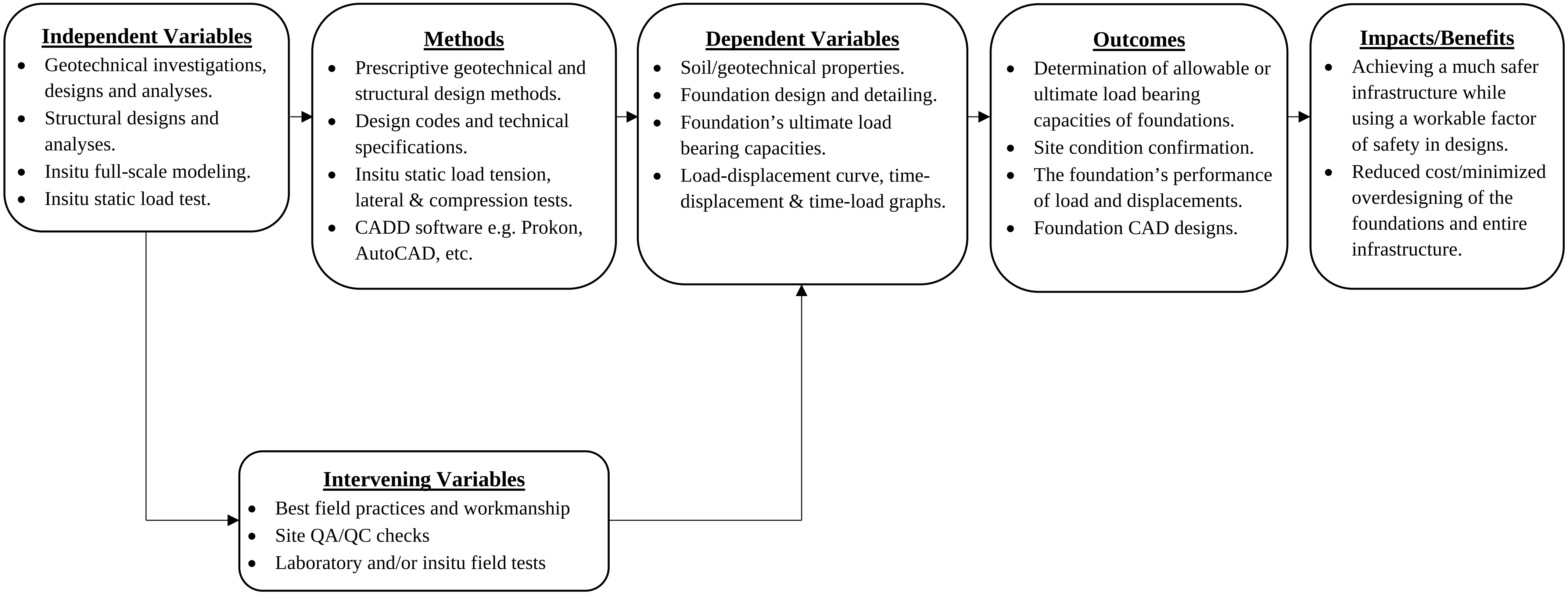


Figure 1.1: Conceptual Framework of Research

CHAPTER TWO

LITERATURE REVIEW

2.1 Background of the study

Foundation design entails that neither the foundation units collapse nor should they induce the overall shear failure of the supporting ground, lest the foundation's post-construction settlement values exceed the permissible tolerances in the codes and specifications (Tomlinson and Boorman, 2001; Mosley *et al.*, 2007; Bayliss and Hardy, 2011; Emuriat, 2017).

The design of foundations consists of proportioning the foundation, mitigating the limit state conditions such as the ultimate limit state properties of loss of static equilibrium of the structure, failure by collapse or by fatigue; and/or the serviceability limit state properties of deflection, cracking, vibration, and deterioration of the foundation structure (Przewłócki *et al.*, 2005; Salgado, 2006; Sivakugan and Braja, 2011; An-Bin and Hai-Sui, 2018).

In order to design against these limit states, analyses are carried out that allow for estimating the limit states, and determining the modulus of deformability of the soil, amount of settlement and internal stresses in the foundations; which results are then used to verify if the related limit states are reached (Rodrigo *et al.*, 2008; McCormac and Csernak, 2012; Emuriat, 2017).

The most commonly known foundation design methods are the Load and Resistance Factor Design (LRFD) method and the Allowable Stress Design (ASD) method, both procedures being based on the Limit States Design (LSD) principles, which provide the boundaries of structural usefulness (Mosley *et al*., 2007; ANSI/AISC 360-10, 2010; Rausche *et al*., 2012).

Whereas the LRFD method attempts to unify the design codes for different construction materials and structural systems with the requirement that the strength provided in design is greater than or at least equal to factored loads acting, the ASD method entails that design loads be compared to the nominal resistance of the system by a factor of safety (Honjo *et al*., 2000; Bilge *et al*., 2011; McCormac and Csernak, 2012).

Many design codes such as the AASHTO, ACI, AISC, and PDCA in the United States, and codes across Canada, Australia, Europe and Africa including Ethiopia and Uganda among many nations, have adopted the LRFD approach over the ASD method in treating soil uncertainties and considering the real behaviour of foundation structures (Coduto, 2001; Foye *et al*., 2006a; Foye *et al*., 2006b; Mosley *et al*., 2007).

Uncertainties in geotechnical models and parameters and their effect have long been recognised (Lacasse and Nadim, 1994; Gilbert and Tang, 1995; Phoon and Kulhawy, 1999; Whitman, 2000; Juang *et al*., 2004; Schuster *et al*., 2008; Zhang *et al*., 2009; Juang *et al*., 2009). Thus, to perform a geotechnical and/or foundation design using the prescriptive design approaches, conservative values of the uncertain soil parameters are often adopted along with an ‘experience-calibrated’ factor of safety (Juang *et al*., 2012; Emuriat, 2017).

While the prescriptive design approach has successfully been used for many decades, it lacks the detailed capacity to render a much more realistic performance and behaviour of geotechnical systems and structures such as foundations especially in the presence of these varying uncertainties (Juang *et al.*, 2012).

With reference to Honjo *et al*. (2000), Foye *et al*. (2006a) and Foye *et al.* (2006b), it was noted that the magnitude of uncertainties involved in the foundation affects the quantity of partial factors assigned to the load and the resistance side of the design, and when compared to the superstructures, foundations and/or geotechnical structures have more unresolved uncertainties in the resistance side than load side (Juang *et al.*, 2012). So, whether the LRFD or ASD approach is used, the goal is to reduce the probability of failure and to obtain a numerical margin between resistance and load resulting in a tolerable small margin of unacceptable structural response (McCormac and Csernak, 2012; Rodrigo *et al*., 2008).

The uncertainties such as the detailed area geology, and human errors during construction works have been captured and/or controlled in this framework through detailed geotechnical laboratory tests, insitu static load tests, strict adherence to the construction methodology and enforcement of the quality assurance/quality control as detailed in clause S2.0 of the Technical Specification for transmission lines.

In the quest to obtain a more rational design, many researchers such as Wu *et al*. (1989), Christian *et al*. (1994), Whitman (2000), Phoon *et al*. (2003a, b), Fenton *et al*. (2005), Najjar and Gilbert (2009), Wang (2011), and Zhang *et al*. (2011) have turned to reliability-based designs such as foundation full-scale models, analyses and foundation tests, which creates a much more-realistic geotechnical and/or foundation outcome and is better in quantifying the uncertainties in soil parameters than the prescriptive design approaches (Juang *et al*., 2012). Therefore, since also soils can soften or harden upon shearing and have a much more complex response than perfect plasticity, the recent focus on using realistic full-scale models (Venkatesh *et al*., 2008; Rodrigo *et al*., 2008), provides important insights into the overall response of foundations (Barvashov *et al*., 2008; Murad *et al*., 2017).

2.2 Uncertainties involved in foundation design

Most foundation design uncertainties originate from both structural and geotechnical aspects such as the uncertainties in estimating load effects, inherent variability of the ground, evaluation of geotechnical material properties, and uncertainties with the degree to which the analysis represents the actual behaviour of the foundation and ground supporting it (Coduto, 2001; Barends, 2011; Seeley and Winfield, 2015; Das and Sobhan, 2018).

It is worth noting that the natural ground variability and evaluation of geotechnical properties usually constitute the greatest uncertainty, commensurate with the complex geological processes involved with the deposition and formation of soil and rock; whereas in contrast, gross errors including human errors or omissions that occur in practice are not quantified or taken into account through safety factors in design. These errors are usually mitigated through quality control and quality assurance programs, and independent third-party reviews on larger projects (Ming and Barreto, 2015; Thounaojam and Sultana, 2015; Sriram and Prasad, 2017).

2.3 General Types of Foundations

A foundation being a critical element of any structure, ought to safely carry the desired loads both structurally and geotechnically, and transfers the structure’s loads to the soil on to which it is resting (Al-Khafaji and Andersland, 1992; Bowles, 1997; Mosley *et al*., 2007; Das and Sobhan, 2018). A properly designed foundation transfers the loads through the soil strata without leading to any abrupt increased settlement under any additional load increment and/or without causing any shear failure of the soil. The settlements and shear failures of the soil may cause damage to the entire structure; and if the soil cannot support the applied loads with a sufficient margin of safety, then settlements of the substructure might occur, thus rendering the structure unsafe and causing either expensive remediation or demolition (Barends, 2011; Bosela *et al*., 2012).

Generally, foundations are classified as either shallow or deep foundations. Shallow foundations include strip, pad and raft foundations meanwhile, deep foundations include piles, piers, diaphragm walls and caissons which are used when construction is to be on soft compressible soil (Mathieson *et al*., 2004; Reese *et al*., 2006) or when the structure is subjected to horizontal loads or moments (Hardy and Spangler, 2007), or whenever foundation settlement is inadmissible or dewatering is to be dispensed with (Bazant, 1979; Terzaghi *et al*., 1996; Murthy, 2007; Salgado *et al*., 2007; Das and Sobhan, 2018; An-Bin and Hai-Sui, 2018).

Shallow foundations have been widely used for overhead power transmission lines because in terms of cost, construction, material, and labour work, they have certain advantages over other foundation types such as piles, piers, caissons, and deep foundations. However, using shallow foundations when there are issues of settlement, irregular ground surface or combined bending and axial loading might cause problems (Bowles, 1995; Coduto, 2001; Bayliss and Hardy, 2011; Ming and Barreto, 2015; Sriram and Prasad, 2017).

2.3.1 Shallow Foundations

Terzaghi (1943) suggested that a foundation is shallow if its depth (D_f) is less than or equal to its width (B) that is, $D_f/B \leq 1$. Al-Khafaji and Andersland (1992) defined shallow foundations as any footing that has a width (B) equal to or greater than the depth at which it is buried (D_f) that is, B $\geq D_f$, and more accurately, a shallow foundation is one with an embedment depth (D_f) equal to or less than four times the foundation width (B) that is, $D_f \leq 4B$. Thus, foundations with depth (D_f) equal to 3 to 4 times their width (B) may be considered shallow. Examples include strip footings and spread footings for both isolated and combined raft or mat foundations (Barends, 2011; Das, 2016; Das and Sobhan, 2018; An-Bin and Hai-Sui, 2018; Das, 2019).

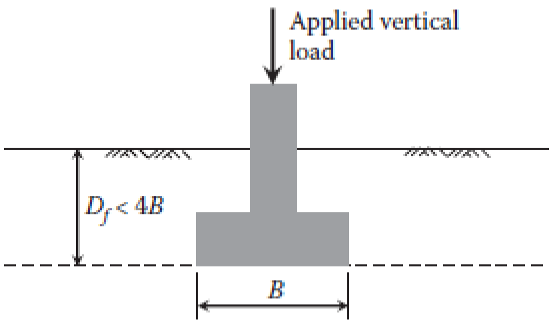


Figure 2.1: A shallow foundation (An-Bin and Hai-Sui, 2018)

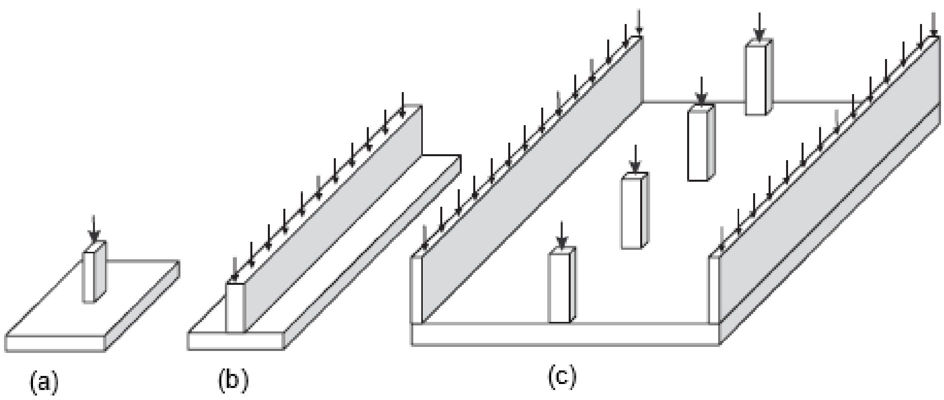


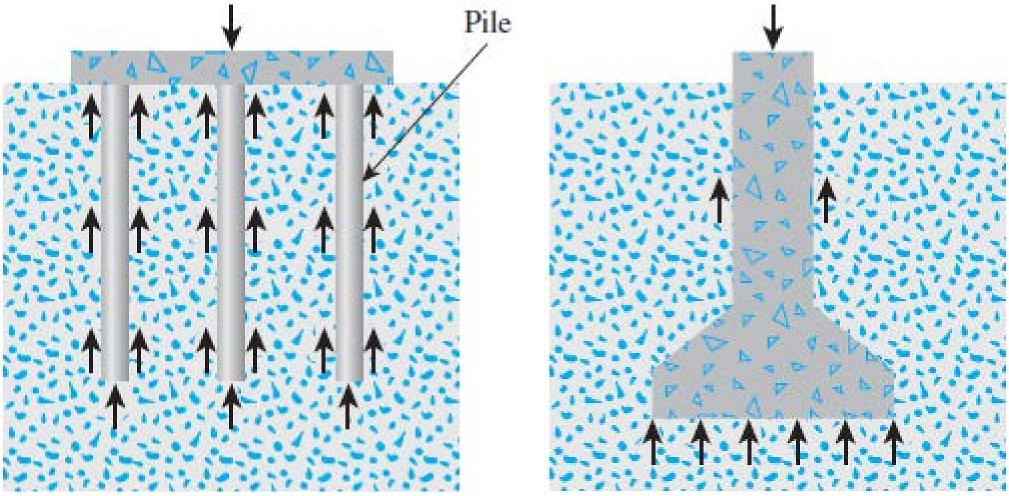
Figure 2.2: Shallow foundations: (a) pad (b) strip (c) raft (Sivakugan, 2011)

2.3.2 Deep Foundations

Deep foundations are those that receive some or all of their support from soil strata at a depth where the embedment depth (D_f) is equal to or greater than four times the foundation width (B) that is, $D_f \geq 4B$ (Das and Sobhan, 2018; An-Bin and Hai-Sui, 2018), and are principally used when weak or otherwise unsuitable soil exists near the ground surface and vertical loads must be carried to strong soils at depth (Murthy, 2007). Deep foundations are also defined as any footing that has a width smaller than the depth to which it extends (Al-Khafaji and Andersland, 1992).

Deep foundations have a number of other uses, such as to resist scour, to sustain axial loading by side resistance in strata of granular soil or competent clay, to allow above-water construction when piles are driven through the legs of a template to support an offshore platform, to serve as breasting and mooring dolphins, and to improve the stability of slopes among special purposes.

The principal deep foundations of driven piles, drilled shafts, caissons and barrettes (Mathieson *et al*., 2004; Reese *et al*., 2006) are structural columns that extend down into soil as either end-bearing if they extend all the way to rock or hard soil, or as friction piles if they are mainly supported by friction along the sides (Ming and Barreto, 2015), and as compaction piles when they are driven into loose sand to densify or increase its bearing capacity (Hardy and Spangler, 2007; Barends, 2011).



(a) (b)

Figure 2.3: Deep foundations: (a) Pile (b) drilled shaft (Das and Sobhan, 2018)

2.4 Transmission Line foundation types

The most common foundations types used on transmission lines in Uganda are the good soil, poor soil, soft rock, hard rock, waterlogged and pile foundations according to the soil conditions, equivalent structure size and load components acting on the foundations (IS 5613-2-2, 1985; IS 1200-1, 1992; Kim and Cho, 1995; IEEE-691, 2001; Jang *et al*., 2007; Bayliss and Hardy, 2011; MoE & MD, 2013; Sriram and Prasad, 2017).

2.4.1 Good Soil Foundations

These types of foundations are Concrete Pyramid/Block type, that is, Plain Cement Concrete (PCC) or Reinforced Cement Concrete (RCC) Pad and Chimney foundations, especially where the soil investigations show good underlying soils at normal foundation depths below poor top soils/silts; and they are cast directly against the edge of the excavation for a minimum height of 350 mm; with the earth frustum assumed to resist uplift and considered to start from the bottom of the vertical edges of the given concrete foundation type in use (MoE & MD, 2013;

Sriram and Prasad, 2017). However, where such a concrete and excavated soil interface cannot be achieved, the frustum is assumed to start from the top of the pad edges (IS 5613-2-2, 1985; IS 1200-1, 1992; IEEE-691, 2001; Jang *et al*., 2007; Bayliss and Hardy, 2011).

2.4.2 Poor Soil Foundations

These types of foundations are used where the soil investigations show that the top layer of "black cotton" soil exceeds 50% or extends up to full depth with the subsoil water/ground water table being far below the formation level of excavation. These foundations exist were the soils are weak or fine-grained, or where water is at the underside of that foundation subject to the extent of any seasonal or monsoonal inundation (IS 5613-2-2, 1985; IS 1200-1, 1992; IEEE-691, 2001; Jang *et al*., 2007; Bayliss and Hardy, 2011; MoE & MD, 2013; Sriram and Prasad, 2017).

2.4.3 Soft Rock Soil Foundations

These types of foundations are generally used for locations where "Soft Rock" occurs for more than the bottom 50% of the "Good Soil" foundation depth.

These "Soft Rocks" include homogeneously weathered rocks, hard rocks which has been fissured and stratified, and/or decomposed rocks. For this foundation type, the size of excavations is no larger than that of "Good Soil" foundations but undercut at the base; the depth of the excavation made at least half the full depth of the excavation required for a "Good Soil" foundation, with the base of the excavation undercut into the soft rock by at least 250 mm all round at an angle between 45° and 60° to the horizontal; and the resistance against sliding created by casting the foundation against the rock surface, for which the excavation is the exact size of the foundation (IS 5613-2-2, 1985; IS 1200-1, 1992; IEEE-691, 2001; Jang *et al.*, 2007; Bayliss and Hardy, 2011; MoE & MD, 2013; Sriram and Prasad, 2017).

2.4.4 Hard Rock Foundations

This foundation type falls under "Guyed tower foundations" and is used where "Hard Rock" is encountered at 1.5 m depth or less below ground level, and/or where the foundation stub legs can be set with a minimum depth of 0.9 m into a concrete block, or where sufficient stub cleats are used to ensure full transfer of load within the foundation. Thus, in hard rock foundations, the upper parts of the stubs are encased in concrete to a height of 300 mm above ground level, and a sufficient number of rebars are grouted into the rock using an expanding grout, for a minimum depth of 1.2 m from the base of the excavation to ensure adequate uplift resistance; whereas, resistance against sliding is generated by casting the foundation against the rock surface, for which excavation shall be the exact size of the foundation, and provisions are made for use of "rock anchor bars" for resistance against uplift (IS 5613-2-2, 1985; IS 1200-1, 1992; IEEE-691, 2001; Jang *et al.*, 2007; Bayliss and Hardy, 2011; MoE & MD, 2013; Sriram and Prasad, 2017).

2.4.5 Waterlogged Ground Foundations

The waterlogged RCC raft type foundations are used where soil investigations show that the locations are either waterlogged, submerged or swampy, or where "black cotton" and other types of soils are subjected to substantially long term or permanent submersion at depths greater than or equal to normal shallow foundation depths (IS 5613-2-2, 1985; IS 1200-1, 1992; IEEE-691, 2001; Jang *et al.*, 2007; Bayliss and Hardy, 2011; Sriram and Prasad, 2017).

2.4.6 Pile Foundations

Pile foundations are principally considered when soils exhibit low bearing capacities, or if they are deep swampy or other types of soil found to be unsuitable for a concrete raft type foundation and/or any of the above forms of waterlogged raft foundation types (IS 5613-2-2, 1985; IEEE-691, 2001; Jang *et al*., 2007). They are often used where there is a concern about excessive settlement of shallow foundations, even though a raft would have an acceptable factor of safety (FoS) in the excesses of 3 against bearing-capacity failure. Hence, it should be noted that these pile foundations usually transmit structural loads through soft and unsuitable upper soil layers to deeper more competent strata, to resist uplift or lateral forces, to support structures over water and carry loads below scour depths (IS 5613-2-2, 1985; IS 1200-1, 1992; IEEE-691, 2001; Jang *et al*., 2007; Bayliss and Hardy, 2011; Sriram and Prasad, 2017).

2.5 Foundation Load and Settlement Testing

Reza and Abdolhosain (2013) noted that the most popular methods for settlement predictions were proposed by Terzaghi and Peck (1967), Schmertmann (1970), Schmertmann *et al*. (1978), Burland *et al*. (1985), and Meyerhof (1956). Reza and Abdolhosain (2013) also stated that various other researchers such as D'Appolonia *et al*. (1970), Berardi and Lancellotta (1991), Sargand *et al*. (1997), Shahin *et al*. (2002), Sivakugan and Johnson (2004), Rasin and Kasktas (2009), and Duzceer (2009), suggested different models and formulae to arrive at the settlement.

Al-Taie *et al*. (2016) divided various methods to calculate the elastic settlement available at the present time into three general categories such as empirical, semi-empirical and theoretical. On the other hand, Das and Sivakugan (2007) stated that, one of the main factors that contribute to the uncertainty in load-settlement predictions is our inability to correctly quantify the soil stiffness, hence, the need for the foundation load test which can validate the computed capacity for a foundation and also provide information for the improvement of design rationale (Poulos and Davis, 1980). There are about three major types of foundation loading tests, namely static load testing, dynamic load testing, and statnamic load testing (Monnet, 2015; Tomlinson and Woodward, 2015).

2.5.1 Static Load Tests

Static Load Tests (SLT) are the most reliable and fundamental forms of in-situ loading tests. They are considered as the bench-mark of foundation performance and used for validating the foundation load capacity and other foundation design assumptions regarding the axial compression or axial tension resistance provided by a foundation element, or its deflected shape under a lateral load. They involve the direct measurement of foundation head displacements in response to a physically applied test load until its failure point to replicate the long-term sustained load conditions (Byrne and Berry, 2008; Monnet, 2015; Tomlinson and Woodward, 2015). These tests are standardised by ASTM D1143 for Standard Test Methods for Deep Foundations Under Static Axial Compressive Load; ASTM D3689 for Standard Test Methods for Deep Foundations Under Static Axial Tensile (or uplift) Load; and ASTM D3966 for Standard Test Methods for Deep Foundations Under Lateral Load (Hertlein and Davis, 2006; ASTM D1143, 2013; ASTM D3689, 2013; ASTM D3966, 2013; Monnet, 2015).

2.5.1.1 Static Axial Compressive Load Test

Static Axial Compressive Load Test measures the axial deflections of vertical or inclined foundations when loaded in static axial compression. This vertical compression maintained-load test is usually carried out to confirm the foundation's structural and geotechnical reliability and to predict its settlement rate. The load is thus, increased in stages until the proposed working load and a certain factor of safety is reached and then unloading the load until the rise or rebound has substantially ceased as per ASTM D1143/D1143M-07 (2013).

The foundation may be tested in three cycles; whereby the first cycle is to 150% of foundation's Design Load (DL), the second cycle test is to 200% of DL and the third cycle tests the foundation to its ultimate load, defined as 250% to 300% of its DL (Hertlein and Davis, 2006; ASTM D1143, 2013). Since the procedure leading up to 300% of the Design Load is very time consuming, the most commonly used method stops at the first cycle up to 150% of the foundation's Design load but may be limited to between 100% to 130% of the design load (IEC 61773, 1996; Byrne and Berry, 2008; Monnet, 2015).

2.5.1.2 Static Axial Tensile Load Test

As standardised by ASTM D3689/D3689M-07 (2013) and COMESA/FDHS 293 (2007), the Static Axial Tensile Load test is used for verifying the behaviour of vertical or batter tension foundations like those of overhead transmission lines with respect to their tensile capacity and axial stiffness, and it provides the most reliable relationship between the static tensile load applied axially to a foundation and the resulting axial movements. Hence, the information obtained are used in assessing the foundation shaft's side shear resistance distribution, amount of end-bearing developed and the long-term load-deflection behaviour. It can also be used to determine if the foundation has an ultimate static capacity and a deflection at service load satisfactory to support a specified foundation or superstructure (Hertlein and Davis, 2006; COMESA/FDHS 293, 2007; ASTM D3689, 2013).

2.5.1.3 Lateral Load Test

As per ASTM D3966/D3966M-07 (2013), the Lateral Load test measures the lateral deflection of a vertical or inclined foundation when subjected to lateral loading, with the results helping in characterising the variation of pile-soil interaction properties such as the coefficient of horizontal subgrade reaction, and the estimation of bending stresses and lateral deflection over the length of the pile for use in the structural design of the pile (Hertlein and Davis, 2006; COMESA/FDHS 293, 2007; ASTM D3966, 2013).

2.5.2 High Strain Dynamic Load Testing (HSDLT)

High Strain Dynamic Load Testing (HSDLT) is a fast and effective method for assessing the foundation's bearing capacity and integrity. It is usually carried out as a supplement to static load tests in corroborating results (Rausche *et al*., 1985; Long, 2007; Basarkar *et al*., 2011). Here, a dynamic load is applied to the pile head using a falling mass while recording acceleration and strain on it. This test procedure is standardised by ASTM D4945-17 Standard Test Method for High Strain Dynamic Testing (HSDT) of Piles, encompassing both Dynamic Pile Monitoring (DPM) and Dynamic Load Testing (ASTM D4945, 2017).

The HSDT method is called Dynamic Pile Monitoring (DPM) when it is applied during pile driving to compute the energy delivered to the pile, compression stresses at the pile top and toe, and tension stresses along the shaft as well as the pile integrity; whereas it is called Dynamic Load Testing (DLT) when it is applied after pile installations regardless of the installation method. It is used in the computing of static capacity and resistance distribution. In addition to the static load capacity of the foundation, dynamic load tests provide data on the force delivered by the pile driving hammer to the pile, maximum driving compressive stresses, structural damage location and extent, resistance distribution, hammer performance, and soil characteristics such as soil damping coefficients and quake values (Hertlein and Davis, 2006; Tomlinson and Woodward, 2015).

2.5.3 Statnamic Loading Test

Statnamic Loading Test is a quasi-static loading test carried out in accordance with ASTM D7383-10 for Standard Test Methods for Axial Compressive Force Pulse (Rapid) Testing of Deep Foundations. Statnamic testing works by accelerating a mass upward that in turn imparts a load onto the foundation pile below the Statnamic device. The load is applied and removed smoothly resulting in load application of 100 to 200 milliseconds. This is 30 to 40 times the duration of dynamic pile load testing. As the duration of the loading is relatively long, piles less than 40m in length remain in compression throughout the test, resulting in negligible stress wave effects and potentially simpler analysis. For foundation design, it is necessary to derive the equivalent static load-displacement curve from the Statnamic data (Middendorp *et al.*, 1992). The simplest form of Statnamic analysis used to obtain equivalent static pile response is known as the unloading point method (Hertlein and Davis, 2006; ASTM D7383-10, 2010; Tomlinson and Woodward, 2015).

2.6 Soil Bearing Capacity Determination

The soil's bearing capacity is its resistive capacity against the applied load. On the other hand, bearing pressure can be defined as the maximum contact pressure between the foundation and the soil before failure (Barends, 2011; Das, 2016).

The ultimate bearing capacity of a soil (q_{ult}) is the theoretical maximum load per unit area of the foundation at which it can be supported without failure. The design or allowable bearing capacity (q_{all}) is the maximum possible loading that can be applied over a unit area in which the soil is safely able to resist instability due to shear failure and without exceeding the maximum tolerable settlement; and it is normally calculated from the ultimate bearing capacity using a factor of safety (Das and Sobhan, 2018; An-Bin and Hai-Sui, 2018; Das, 2019).

There are several methods used for determining the bearing capacity of the soil (Das and Sobhan, 2018), but the commonest under limit equilibrium are Terzaghi's bearing capacity (Terzaghi, 1943), Meyerhof's bearing capacity theories (Meyerhof, 1951 and 1963), and Jørgen Brinch Hansen's bearing capacity equations (Hansen, 1970).

2.6.1 Terzaghi's bearing capacity method

Terzaghi (1943) expanded the limit equilibrium theory from two model tests and showed that there are three modes of failure in soil that limit bearing capacity, namely the general shear, local shear, and punching shear failures (Das, 2016; Das and Sobhan, 2018) as shown in Figure 2.4 below. Meanwhile, Table 2.1 presents a summary of the types of bearing capacity failures that would most likely develop, based on soil type and soil properties.

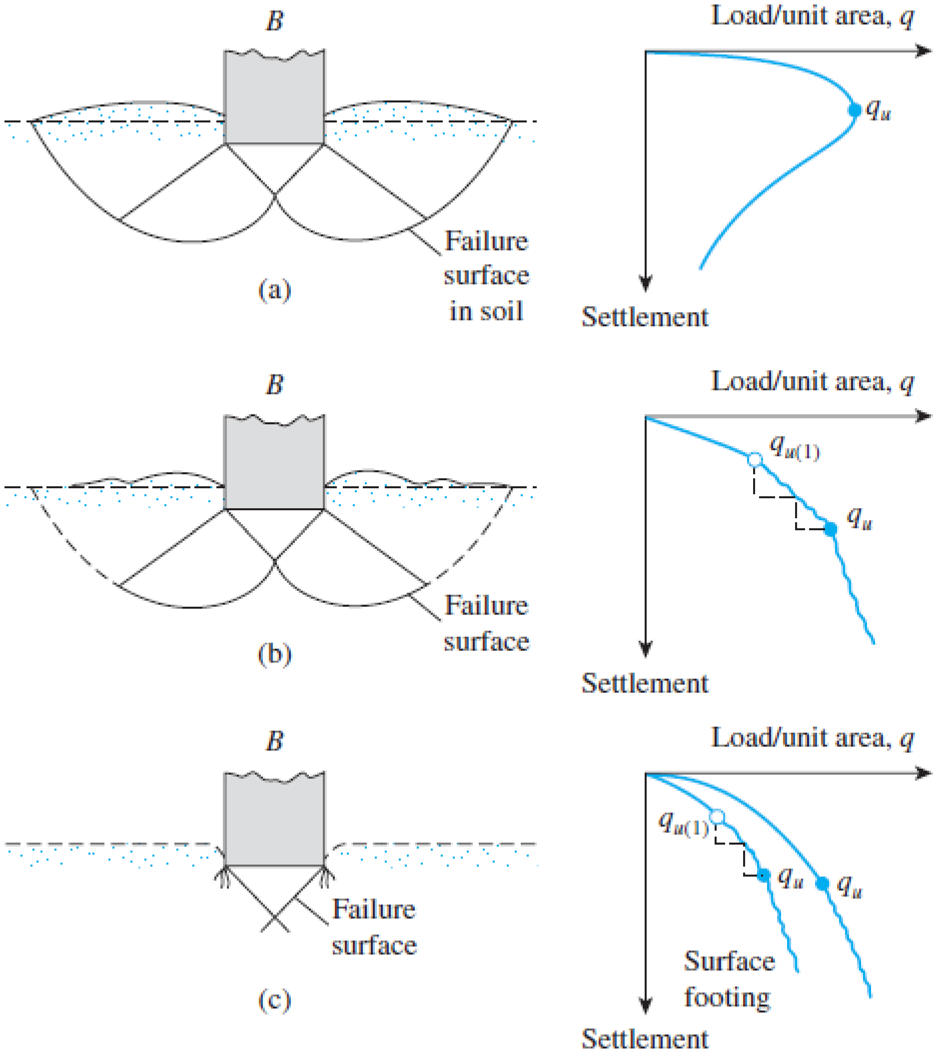


Figure 2.4: Nature of bearing capacity failure in soil (Das, 2016)

Table 2.1: Bearing capacity failure (Vesic, 1963 and 1973)

	Cohesionless soil (e.g. sand)			Cohesive soil (e.g. clays)	
Bearing capacity failure	**Density condition**	**Relative density**	$(N_1)_{60}$	**Soil Consistency**	**Undrained shear strength**
General shear (Fig. 2.4a)	Dense to very dense	65-100%	> 20	Very stiff to hard	> 100 kPa
Local shear (Fig. 2.4b)	Medium	35-65%	5-20	Medium to stiff	25-100 kPa
Punching shear (Fig. 2.4c)	Loose to very loose	0-35%	< 5	Soft to very soft	< 25 kPa

Where: $(N_1)_{60}$ is the corrected Standard Penetration Test (SPT) value.

2.6.1.1 General shear failure case

This type of failure is normally seen in dense and stiff soils. In design, failures in this mode are handled by equations 2.1 to 2.4 that account for soil cohesion, friction, embedment, surcharge, and self-weight, as given below for square, rectangular, continuous and circular footings respectively (Das, 2016):

$$q_{ult} = 1.3c'N_c + qN_q + 0.4\gamma' BN_\gamma \tag{2.1}$$

$$q_{ult} = \left(1 + 0.3\frac{B}{L}\right)c'N_c + qN_q + \left(1 - 0.2\frac{B}{L}\right)0.5\gamma' BN_\gamma \tag{2.2}$$

$$q_{ult} = c'N_c + qN_q + 0.5\gamma' BN_\gamma \tag{2.3}$$

$$q_{ult} = 1.3c'N_c + qN_q + 0.3\gamma' BN_\gamma \tag{2.4}$$

Where:

q_{ult} = ultimate bearing capacity of the foundation

$q = \gamma D_f =$ unit surcharge; and $\gamma' =$ effective unit weight of soil

$D_f =$ depth of foundation; $c' = \frac{2}{3}c =$ effective cohesion; and $c =$ cohesion

$N_q, N_c,$ and $N_\gamma =$ Terzaghi's bearing capacity factors

$N_q = \frac{e^{2\pi(0.75-\emptyset'/360)\tan\emptyset'}}{2\,cos^2(45+\emptyset'/2)}$; $N_c = 5.14$ for $\emptyset' = 0$; and $N_c = \frac{N_q-1}{\tan\emptyset'}$ for $\emptyset' > 0$

$N_\gamma = \frac{\tan\emptyset'}{2}\left(\frac{K_{P\gamma}}{cos^2\emptyset'} - 1\right)$; and $B =$ width or the diameter of the foundation

$\emptyset' =$ effecive internal friction angle, and

$K_{P\gamma} =$ passive pressure coefficient

However, simplifications by Coduto (2001) eliminates the use of $K_{P\gamma}$, and gives accurate values to within 10% when the simplified (N_γ) below is used:

$$N_\gamma = \frac{2\left(N_q + 1\right)\tan\emptyset'}{1 + 0.4\sin 4\emptyset'}$$

2.6.1.2 Local shear failure case

This type of failure is normally seen in relatively loose and soft soils; thus, for foundations that exhibit this failure mode in soils, Terzaghi (1943) suggested the following modifications to the previous equations 2.1 to 2.4 for square, rectangular, continuous and circular footings respectively (Das, 2016):

$$q_{ult} = 0.867c'N'_c + qN'_q + 0.4\gamma' BN'_\gamma \quad (2.5)$$

$$q_{ult} = \left(1 + 0.3\frac{B}{L}\right)0.867c'N_c + qN_q + \left(1 - 0.2\frac{B}{L}\right)0.5\gamma' BN_\gamma \quad (2.6)$$

$$q_{ult} = \frac{2}{3}c'N'_c + qN'_q + 0.5\gamma' BN'_\gamma \quad (2.7)$$

$$q_{ult} = 0.867c'N'_c + qN'_q + 0.3\gamma' BN'_\gamma \quad (2.8)$$

The values of Modified $N'_c, N'_q,$ and N'_γ, are calculated using the equations for $N_q, N_c,$ and N_γ, respectively (Das, 2007) by replacing the effective internal angle of friction ($\emptyset'$) by a value equal to: $\tan^{-1}\left(\frac{2}{3}\tan \emptyset'\right)$.

Table 2.2: Terzaghi's Bearing Capacity Factors (Das and Sobhan, 2018)

$\emptyset'$	N_c	N_q	$N_\gamma{}^a$	$\emptyset'$	N_c	N_q	$N_\gamma{}^a$	$\emptyset'$	N_c	N_q	$N_\gamma{}^a$
0	5.70*	1.00	0.00	17	14.60	5.45	2.18	34	52.64	36.50	38.04
1	6.00	1.10	0.01	18	15.12	6.04	2.59	35	57.75	41.44	45.41
2	6.30	1.22	0.04	19	16.56	6.70	3.07	36	63.53	47.16	54.36
3	6.62	1.35	0.06	20	17.69	7.44	3.64	37	70.01	53.80	65.27
4	6.97	1.49	0.10	21	18.92	8.26	4.31	38	77.50	61.55	78.61
5	7.34	1.64	0.14	22	20.27	9.19	5.09	39	85.97	70.61	95.03
6	7.73	1.81	0.20	23	21.75	10.23	6.00	40	95.66	81.27	115.31
7	8.15	2.00	0.27	24	23.36	11.40	7.08	41	106.81	93.85	140.51
8	8.60	2.21	0.35	25	25.13	12.72	8.34	42	119.67	108.75	171.99
9	9.09	2.44	0.44	26	27.09	14.21	9.84	43	134.58	126.50	211.56
10	9.61	2.69	0.56	27	29.24	15.90	11.60	44	151.95	147.74	261.60
11	10.16	2.98	0.69	28	31.61	17.81	13.70	45	172.28	173.28	325.34
12	10.76	3.29	0.85	29	34.24	19.98	16.18	46	196.22	204.19	407.11
13	11.41	3.63	1.04	30	37.16	22.46	19.13	47	224.55	241.80	512.84
14	12.11	4.02	1.26	31	40.41	25.28	22.65	48	258.28	287.85	650.67
15	12.86	4.45	1.52	32	44.04	28.52	26.87	49	298.71	344.63	831.99
16	13.68	4.92	1.82	33	48.09	32.23	31.94	50	347.50	415.14	1072.80

Note: [a] The $N_\gamma{}^a$ values are from Kumbhojkar (1993);
* $N_c = 1.5\pi + 1$ [See Terzaghi (1943), pg. 127 (Bowles, 1997)], and
The values for N_γ for ϕ of 0°, 34°, and 48° are original Terzaghi values and used to back-compute $K_{p\gamma}$.

Table 2.3: Terzaghi's Modified Bearing Capacity Factors (Das, 2017)

$\emptyset'$	N'_c	N'_q	N'_γ	$\emptyset'$	N'_c	N'_q	N'_γ	$\emptyset'$	N'_c	N'_q	N'_γ
0	5.70	1.00	0.00	17	10.47	3.13	0.76	34	23.72	11.67	7.22
1	5.90	1.07	0.005	18	10.90	3.36	0.88	35	25.18	12.75	8.35
2	6.10	1.14	0.02	19	11.36	3.61	1.03	36	26.77	13.97	9.41
3	6.30	1.22	0.04	20	11.85	3.88	1.12	37	28.51	15.32	10.90
4	6.51	1.30	0.055	21	12.37	4.17	1.35	38	30.43	16.85	12.75
5	6.74	1.39	0.074	22	12.92	4.48	1.55	39	32.53	18.56	14.71
6	6.97	1.49	0.10	23	13.51	4.82	1.74	40	34.87	20.50	17.22
7	7.22	1.59	0.128	24	14.14	5.20	1.97	41	37.45	22.70	19.75
8	7.47	1.70	0.16	25	14.80	5.60	2.25	42	40.33	25.21	22.50
9	7.74	1.82	0.20	26	15.53	6.05	2.59	43	43.54	28.06	26.25
10	8.02	1.94	0.24	27	16.30	6.54	2.88	44	47.13	31.34	30.40
11	8.32	2.08	0.30	28	17.13	7.07	3.29	45	51.17	35.11	36.00
12	8.63	2.22	0.35	29	18.03	7.66	3.76	46	55.73	39.48	41.70
13	8.96	2.38	0.42	30	18.99	8.31	4.39	47	60.91	44.45	49.30
14	9.31	2.55	0.48	31	20.03	9.03	4.83	48	66.80	50.46	59.25
15	9.67	2.73	0.57	32	21.16	9.82	5.51	49	73.55	57.41	71.45
16	10.06	2.92	0.67	33	22.39	10.69	6.32	50	81.31	65.50	85.75

2.6.1.3 Punching shear failure case

This failure type is normally seen in relatively loose sand with relative density less than 35% or clays of soft consistency (loose and soft soils), soil of very high compressibility and in deeper elevations. Punching shear design is often undertaken in structural design calculations not under geotechnical designs (Das, 2016).

2.6.2 Meyerhof's bearing capacity method

Meyerhof (1951, 1963) proposed two general equations for bearing capacity calculation similar to Terzaghi's but introducing further foundation shape coefficients (s_q) that multiplies the N_q factor, depth factors $(d_i)$and inclination factors (i_i) for cases where the load line is inclined to the vertical.

2.6.2.1 Meyerhof's equations

For the case when the resultant load at the bearing level (Q_b) is vertical with no horizontal components, then Meyerhof's vertical load equation is:

$$Q_b = cN_cS_cd_c + q_oN_qS_qd_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma \quad (2.9)$$

For the case when the resultant load at the bearing level (Q_b) is inclined from vertical and can be resolved into vertical and horizontal components, with the horizontal component of load in the direction of the width of the footing, then Meyerhof's Inclined Load equation is given as:

$$Q_b = cN_cd_ci_c + q_oN_qd_qi_q + 0.5\gamma BN_\gamma d_\gamma i_\gamma \quad (2.10)$$

Where:

$\mathrm{Q_b}$ = the resultant load at the bearing level; and c = cohesion of soil

q_o = the surcharge pressure; and γ = unit weight of soil

$\mathrm{N_c}$, $\mathrm{N_q}$, and $\mathrm{N_\gamma}$ = Meyerhof′s bearing capacity factors

$\mathrm{N_q} = e^{(\pi \tan \emptyset)}\, tan^2(45 + \emptyset/2)$; and $\mathrm{N_c} = \cot \emptyset\ (\mathrm{N_q} - 1)$

$\mathrm{N_\gamma} = (\mathrm{N_q} - 1)\tan(1.4\emptyset)$; $\mathrm{S_c}$, $\mathrm{S_q}$, and $\mathrm{S_\gamma}$ = Shape factors;

$\mathrm{d_c}$, $\mathrm{d_q}$, and $\mathrm{d_\gamma}$ = Depth factors; and $\mathrm{i_c}$, $\mathrm{i_q}$, and $\mathrm{i_\gamma}$ = Inclined load factors

Table 2.4: Meyerhof's factors vs friction angle (Meyerhof, 1951 and 1963)

Friction Angle	Shape Factor	Depth Factor	Inclined Load Factors
Any ϕ	$\mathrm{S_c} = 1 + 0.2K_p\left(\frac{B}{L}\right)$	$\mathrm{d_c} = 1 + 0.2\sqrt{K_p}\left(\frac{D}{B}\right)$	$\mathrm{i_c} = (1 - \theta/90^\circ)^2$ $\mathrm{i_q} = (1 - \theta/90^\circ)^2$
ϕ = 0	$\mathrm{S_q} = \mathrm{S_\gamma} = 1$	$\mathrm{d_q} = \mathrm{d_\gamma} = 1$	$\mathrm{i_\gamma} = 1$
ϕ ≥ 10°	$\mathrm{S_q} = 1 + 0.1K_p\left(\frac{B}{L}\right)$ $\mathrm{S_\gamma} = 1 + 0.1K_p\left(\frac{B}{L}\right)$	$\mathrm{d_q} = 1 + 0.1\sqrt{K_p}\left(\frac{D}{B}\right)$ $\mathrm{d_\gamma} = 1 + 0.1\sqrt{K_p}\left(\frac{D}{B}\right)$	$\mathrm{i_\gamma} = (1 - \theta/\emptyset)^2$

Where:

D = depth of the footing; B = width of the footing

L = length of the footing; and $\emptyset$ = soil friction angle

$K_p = \tan^2(45 + \emptyset/2)$ = passive pressure coefficient

$\theta = \tan^{-1}(Q_h/Q_v)$ = angle of the load in degrees

In the above equations involving B and L, for eccentricity case at the bearing elevation, B_{eff} is used instead of B [where $\left(B_{eff} = B - 2e_B\right)$], and L_{eff} instead of L [where $\left(L_{eff} = L - 2e_L\right)$]. However, in unusual cases where $(L - 2e_L) < (B - 2e_B)$, use $\left(B_{eff} = L - 2e_L\right)$ and $\left(L_{eff} = B - 2e_B\right)$.

2.6.3 Brinch Hansen's bearing capacity method

Brinch Hansen (1970) provided equations to estimate limit bearing capacity for two separate cases of strength parameters namely that ϕ > 0 (Case 1), and ϕ = 0 (Case 2) for undrained clay (Das, 2007; Das and Sobhan, 2018).

In addition, for each of these cases there are two separate subcases:

i) Either no horizontal load component (Q_{tr}) or there is a horizontal load component and it is in the direction of the width of the footing $\left(Q_{tr,B}\right)$.

ii) There is a horizontal load component in the direction of the length of the footing $\left(Q_{tr,L}\right)$, or in both directions of width and length of the footing $\left[\left(Q_{tr,B}\right)\text{and }\left(Q_{tr,L}\right)\right]$.

In all cases, the limit load that can be carried at the bearing level is given by:

$$Q_{bL} = q_{bL}\ x\ A_f \qquad (2.11)$$

a) Case 1 (ϕ > 0)

For Subcase (a) where there is no (Q_{tr}) or where there is only $\left(Q_{tr,B}\right)$:

$$q_{bL} = cN_cS_cd_ci_cb_cg_c + q_oN_qS_qd_qi_qb_qg_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \qquad (2.12)$$

Where:

$q_o =$ effective stress at the bearing level for an effective stress analysis, or it is the total stress at the bearing level for a total stress analysis.

$\mathrm{N_c, N_q,}$ and $\mathrm{N_\gamma}$ = The same as Meyerhof's; $\mathrm{S_c, S_q,}$ and $\mathrm{S_\gamma}$ = Shape factors

$\mathrm{d_c, d_q,}$ and $\mathrm{d_\gamma}$ = Depth factors; $\mathrm{i_c, i_q,}$ and $\mathrm{i_\gamma}$ = Inclined load factors

$\mathrm{g_c, g_q,}$ and $\mathrm{g_\gamma}$ = Ground factors; $\mathrm{N_q} = e^{(\pi \tan \emptyset)}\, tan^2(45 + \emptyset/2)$;

$\mathrm{N_c} = \cot\emptyset\ (\mathrm{N_q} - 1)$; and $\mathrm{N_\gamma} = (\mathrm{N_q} - 1)\tan(1.4\emptyset)$

$\mathrm{S_c} = 1 + \cos\emptyset\left(\frac{N_q}{N_c}\ x\,\frac{B}{L}\right)$; $\mathrm{S_q} = 1 + \sin\emptyset\left(\frac{B}{L}\right)$; and $\mathrm{S_\gamma} = 1 - 0.4\left(\frac{B}{L}\right) \geq 0.6$

$$\mathrm{d_c} = 1 + 2\,(1 - \sin\emptyset)^2\left(\frac{N_q}{N_c}\ x\,\frac{D}{B}\right)\ \text{for}\ \frac{D}{B} \leq 1$$

$$\mathrm{d_c} = 1 + 2\,(1 - \sin\emptyset)^2\left(\frac{N_q}{N_c}\right)\tan^{-1}\left(\frac{D}{B}\right)\ \text{for}\ \frac{D}{B} > 1$$

$$\mathrm{d_q} = 1 + 2\tan\emptyset\ (1 - \sin\emptyset)^2\left(\frac{D}{B}\right)\ \text{for}\ \frac{D}{B} \leq 1$$

$\mathrm{d_q} = 1 + 2\tan\emptyset\ (1 - \sin\emptyset)^2\tan^{-1}\left(\frac{D}{B}\right)$ for $\frac{D}{B} > 1$; and $\mathrm{d_\gamma} = 1$

$\mathrm{i_q} = \left(1 - \frac{0.5\,Q_{tr}}{Q_{ax} + A\,C_a\ \cot\emptyset}\right)^{0.5} \geq 0$; and $\mathrm{i_\gamma} = \left(1 - \frac{0.7\,Q_{tr}}{Q_{ax} + A\,C_a\ \cot\emptyset}\right)^{0.5} \geq 0$

$$\mathrm{i_c} = \left[\mathrm{i_q} - \frac{1 - \mathrm{i_q}}{N_q - 1}\right]$$

$\mathrm{b_q} = \mathrm{e}^{-[0.0349066\ \mathrm{x}\ a_b \tan\emptyset]}$; and $\mathrm{b_\gamma} = \mathrm{e}^{-[0.0471239\ \mathrm{x}\ a_b \tan\emptyset]}$

$\mathrm{g_q} = \mathrm{g_\gamma} = \left[1 - 0.5\tan(a_g)\right]^5$; and $\mathrm{g_c} = \left[\mathrm{g_q} - \frac{1 - \mathrm{g_q}}{N_q - 1}\right]$

b) Case 2 (ϕ = 0)

For Subcase (a) where there is no (Q_{tr}) or where there is only $(Q_{tr,B})$:

$$q_{bL} = (\pi + 2)S_u(1 + S_{su} + d_{su} - i_{su} - b_{su} - g_{su}) + q_o \tag{2.13}$$

Where:

$q_o =$ total effective stress at the bearing level

$S_{su} = 0.2\left(\frac{B}{L}\right)$; and $d_{su} = 0.4\left(\frac{D}{B}\right)$ for $D \leq B$

$d_{su} = 0.4\tan^{-1}\left(\frac{D}{B}\right)$ for $D > B$; and $i_{su} = 0.5 - 0.5\sqrt{1 - \frac{Q_{tr} - B}{C_a\,A_f}}$

$C_a =$ adhesive stress acting on base of footing in the range of 0.5 to 1.0 of S_u.

For a rough base, $C_a = S_u$.

$$b_{su} = \frac{2\,a_b\ (rad)}{\pi + 2} = \frac{a_b\ (degrees)}{147.3}\text{; and } g_{su} = \frac{2\,a_g\ (rad)}{\pi + 2} = \frac{a_g\ (degrees)}{147.3}$$

Note:

- For both Cases 1 and 2 above under Subcase (b), where there is only $(Q_{tr,L})$ or both $(Q_{tr,B})$ and $(Q_{tr,L})$; the design engineer ought to check for q_{bL} separately in the directions of the width and length of footing, noting that q_{bL} is equal to the smaller of the two values.
- Also in the above equations involving B and L, for cases of eccentricity at the bearing elevation, B_{eff} is used instead of B [where $B_{eff} = B - 2e_B$], and L_{eff} is used instead of L [where $L_{eff} = L - 2e_L$].

  However, in unusual cases where $(L - 2e_L) < (B - 2e_B)$, one must use $B_{eff} = L - 2e_L$ and $L_{eff} = B - 2e_B$.
- Although Hansen's equation considers issues of base tilting and footings on slopes, can be used for both shallow and deep foundations, and gives better correlation than the other methods in full-scale footing tests, Terzaghi's is the most widely used and preferred method among many geotechnical engineers because of its simplicity and it gives the exact solution without superposition approximation, in which the bearing capacity factor is dependent on the dimensionless parameter (λ) and the friction angle $(\emptyset)$ (Sun *et al*., 2013).

2.6.4 Allowable bearing capacity evaluation from SPT Test

2.6.4.1 Terzaghi's approach

The allowable bearing capacities, q_{all} can be computed using the corrected SPT N'_{55} values from Terzaghi's formula (1967) for cohesive soils.

The following assumptions are normally made in the calculation of bearing capacity based on corrected SPT N-values:

- The Peck et al. (1967) relationship between N-values and unconfined compressive strength is valid (Published by Terzaghi and peck, 1967).
- The maximum allowable settlement in cohesive soils is 25 mm.
- The design N-values are derived from the statistical average of all values within a depth zone equal to the footing width below the founding depth.

The equations used to evaluate the bearing capacity for cohesive soils are:

- Unconfined compressive strength (q_u); $q_u = 13.1 \text{ x corrected N} - \text{value}$
- Corrected N-value (N'_{55}); $N'_{55} = C_N \text{ x N x } \eta_1 \text{ x } \eta_2 \text{ x } \eta_3 \text{ x } \eta_4$
- Undrained Cohesion (c_u); $c_u = {}^{q_u}/_2$
- Ultimate bearing capacity (q_{ult}); $q_{ult} = 5.14 \text{ x } c_u$
- Factor of Safety (FS); FS = 3.0
- Allowable bearing capacity $(q_{all}) = \frac{q_{ult}}{FS} = \frac{\text{Ultimate bearing capacity}}{FS}$
- $C_N = \text{adjustment for overburden pressure}, \left(\frac{p''_o}{p'_o}\right)^{\frac{1}{2}}$

The adjustment for effective overburden pressure (C_N) is normally computed using Liao and Whitman's formula (1986) below (Martin and Lew, 1999):

- $C_N = \left(\frac{p''_o}{p'_o}\right)^{\frac{1}{2}} = \left(\frac{95.76}{p'_o}\right)^{\frac{1}{2}} \text{ where } 0.4 \le C_N \le 1.7$

Where:

- p''_o = reference overburden pressure (95.76 KPa or 1.0 kg/cm^2)
- p'_o = overburden pressure = (Effective unit weight x depth)
- $\eta_1 = E_r/E_{rb}$; and γ' = effective unit weight of soil
- E_r = the average energy ratio that depends on the drill system = 45
- E_{rb} = the standard energy ratio = 55; η_2 = rod length correction
- η_3 = sampler correction; and η_4 = borehole diameter correction

In the geotechnical design calculations of bearing capacity from SPT tests, the following equations are essential namely:

$$\Rightarrow \mathrm{N'}_{55} = \left(\frac{\mathrm{p''}_{\mathrm{o}}}{\gamma' \text{ x depth}}\right)^{\frac{1}{2}} \text{x N x } \frac{\mathrm{E_r}}{\mathrm{E_{rb}}} \text{ x } \eta_2 \text{ x } \eta_3 \text{ x } \eta_4 \quad (2.14)$$

$$\Rightarrow \mathrm{q_{ult}} = 5.14 \text{ x} \frac{\mathrm{q_u}}{2} = \left[5.14 \text{ x} \frac{13.1 \text{ x N'}_{55}}{2}\right] \quad (2.15)$$

$$\Rightarrow \mathrm{q_{all}} = \frac{\mathrm{q_{ult}}}{\mathrm{FS}} = (5.14 \text{ x c}_\mathrm{u})/FS = \left\{5.14 \text{ x} \left[\frac{(13.1 \text{ x N'}_{55})}{2}\right]\right\}/FS \quad (2.16)$$

However, for field SPT N-values greater than 50, an N-value of 80 is normally assumed in N_{55} computation (Bowles, 1997; Bryne and Berry, 2008).

2.6.4.2 Bowles's approach based on Meyerhof (1963)

Although, there are several reliable methods for estimating the soil's bearing capacity from the SPT tests, one of the most commonly used method is that of Bowles (1997) using corrected SPT $\mathrm{N'}_{55}$ values to calculate the allowable bearing capacities (q_{all}) as shown below:

$$\mathrm{q_a} = \left\{\frac{\mathrm{N}}{\mathrm{F_2}}\left[\frac{(\mathrm{B}+\mathrm{F_3})}{\mathrm{B}}\right]^2 x\ \mathrm{K_d}\right\} \text{for B} > \mathrm{F_4} \quad (2.17)$$

$$\mathrm{q_a} = \left\{\frac{\mathrm{N}}{\mathrm{F_1}} x\ \mathrm{K_d}\right\} \text{for B} \leq \mathrm{F_4} \quad (2.18)$$

Where:

N = Corrected SPT $\mathrm{N'}_{55}$ values; and $\mathrm{N'}_{55}$ = adjusted N − values

B = Width of foundation; and D = Depth of foundation

$\mathrm{q_a} = q_{all}$ = Allowable bearing pressure for settlement limited to 25 mm

$$\mathrm{K_d} = 1 + \frac{0.33\mathrm{D}}{\mathrm{B}} < 1.33;\ \mathrm{F_1} = 0.05;\ \mathrm{F_2} = 0.08;\ \mathrm{F_3} = 0.3;\ \text{and } \mathrm{F_4} = 1.2$$

The N-values are usually converted to $\mathrm{N'}_{55}$ standard energy ratio value according to Bowles (Bowles, 1997) using the equation below:

$$\mathrm{N'}_{55} = \mathrm{C_N} \times \mathrm{N} \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4 \tag{2.19}$$

Where:

$\mathrm{C_N}$ = adjustment for overburden pressure (Liao and Whitman, 1986)

$$\Rightarrow \mathrm{C_N} = \left(\frac{\mathrm{p''}_o}{\mathrm{p'}_o}\right)^{\frac{1}{2}} = \left(\frac{95.76}{\mathrm{p'}_o}\right)^{\frac{1}{2}} \text{ where } 0.4 \le \mathrm{C_N} \le 1.7$$

$\mathrm{p'}_o$ = overburden pressure

$\mathrm{p''}_o$ = reference overburden pressure (95.76 KPa or 1.0 kg/cm^2)

$\eta_1 = \mathrm{E_r}/\mathrm{E_{rb}}$

$\mathrm{E_r}$ = the average energy ratio that depends on the drill system = 45

$\mathrm{E_{rb}}$ = the standard energy ratio = 55

η_2 = rod length correction; η_3 = sampler correction (1.00 in our case)

η_4 = borehole diameter correction (1.00 in our case)

Bowles's (1982) approach is based on Meyerhof's (1963) equations to evaluate bearing capacities from SPT results, and it uses an increase of 50% basing on the accumulation of field observations to compute the foundation's allowable bearing capacity (q_{all}), based on a unit breadth as shown below:

$$\mathrm{q_{all}} = 0.73 \times \mathrm{N''} \times \mathrm{R_{D_1}} \times \mathrm{S_a} \; [\mathrm{kN/m^2} \text{ for } \mathrm{B} \le 1.2\mathrm{m}] \tag{2.20}$$

$$\mathrm{q_{all}} = 0.48 \times \mathrm{N''} \times \mathrm{R_{D_2}} \times \left(\frac{\mathrm{B} + 0.3}{\mathrm{B}}\right)^2 \times \mathrm{S_a} \; [\text{for } \mathrm{B} > 1.2\mathrm{m}] \tag{2.21}$$

$$\mathrm{R_{D_1}} = 1 + 0.2\left(\frac{\mathrm{D_f}}{\mathrm{B}}\right) \le 1.2 \text{ for } \emptyset = 0 \tag{2.22}$$

$$\mathrm{R_{D_2}} = 1 + 0.1\left(\frac{\mathrm{D_f}}{\mathrm{B}}\right) \le 1.2 \text{ for } \emptyset = 0 \tag{2.23}$$

Where:

$\mathrm{N''}$ = Corrected SPT N-value; and S_a = Allowable settlement (25 mm)

$\mathrm{R_{D_1}}$ and $\mathrm{R_{D_2}}$ = Depth reduction factors (Meyerhof's depth factor)

$\mathrm{D_f}$ = Foundation depth (or test depth in metres)

B = Foundation Breadth (in metres); and

$\emptyset$ = Internal angle of soil friction in degrees

For the description of the consistency of fine-grained soils and relative strength of coarse-grained soils relative to SPT N-values for blows per 300 mm, and/or the need for adjustments factors in the computation of the N'_{55} (adjusted SPT N-values), the tables below may be used in the interpretations thereof:

Table 2.5: Consistency table for fine-grained soils (BS 5930: 1999)

Description	Unconfined Compressive Strength (kPa)	N-value
Very soft	Less than 25	Less than 2
Soft	25 to 50	2 to 5
Firm	50 to 100	5 to 10
Stiff	100 to 200	10 to 20
Very stiff	200 to 380	20 to 40
Hard	Over 380	Over 40

Table 2.6: Relative strength of coarse-grained soils (BS 5930: 1999)

S/No.	Description	N-value
1	Very loose	Less than 4
2	Loose	4 to 10
3	Compact/Medium-dense	10 to 30
4	Dense	30 to 50
5	Very Dense	Over 50

Table 2.7a: Adjustment factors for corrected N'_{55} values (Bowles, 1997)

Hammer Efficiency (%) for η_1 (Average Energy Ratio, E_r)				
Country	**Donut**		**Safety**	
	Rope-Pulley	**Trip**	**Rope-Pulley**	**Trip/Auto**
North America	45	-	70-80	80-100
Japan	67	78	-	-
United Kingdom	-	-	50	60
China/Africa	50	60	-	-

Standard Energy Ratio, E_{rb}	
E_{rb}	**Reference**
50 to 55 (Use 55)	Schmertmann [in Robertson *et al*. (1983)]
60	Seed *et al*. (1985); Skempton (1986)
70 to 80 (Use 70)	Riggs (1986)

Rod Length corrections, η_2			
Length	η_2	**Length**	η_2
More than 10m	1.00	4 - 6m	0.85
6 - 10m	0.95	0 - 4m	0.75

Sample corrections, η_3			
With Liner	η_3	**Without Liner**	η_3
Dense sand	0.80	All soil types	1.00
Clay	0.80		
Loose sand	0.90		

Table 2.7b: Adjustment factors for corrected N'_{55} values (Bowles, 1997)

Borehole diameter corrections, η_3	
Hole diameter	η_3
50 - 120 mm	1.00
150 mm	1.05
200 mm	1.15

2.6.5 Soil bearing capacity evaluation based on DPL test

2.6.5.1 The Dutch Formula

The most common formula for calculating the soil's dynamic point resistance and/or soil bearing capacity (q) and resistance value ($\mathrm{r_d}$) from the Dynamic Penetration Light test (DPL) is the Dutch formula (Sanglerat, 1972; Atkinson, 2004; Khodaparast *et al.*, 2015) as given below:

$$\mathrm{q} = r_d\left[\frac{M}{(M+P)}\right] = \left[\frac{\mathrm{E}}{\mathrm{A \times H}}\right] \times \left[\frac{\mathrm{M}}{(\mathrm{M+P})}\right] \qquad (2.24)$$

Where:

- q = the Soil bearing capacity
- E = the energy in Joules $\Rightarrow \mathrm{E = Mgh}$

$\Rightarrow \mathrm{E} = \text{Hammer mass x gravitational acceleration x falling height}$

- A = the area of the cone = 0.001 m^2
- H = depth of penetration (m);
- M = mass of hammer = 10.252 kg

- P = the mass of the assembly in kilograms; whereby the mass of DPL assembly is 6.714 kg, and mass of each rod is 2.86 kg.

$\Longrightarrow \mathrm{P} = (\text{Mass of Anvil}) + (\text{Mass of rods})$

$$\mathrm{r_d} = \frac{\mathrm{M_1gh}}{\mathrm{A\,e}} = \left[\frac{\mathrm{M_1gh}}{A\,x\left(\frac{0.1}{N_{10}}\right)}\right] = \frac{E}{A\,x\,H} \tag{2.25}$$

Where:

- M_1 = mass of the hammer; and N_{10} = blows per 10 cm penetration
- e = penetration rate = $0.1/N_{10}$
- r_d = unit point resistance; and q_d = dynamic point resistance

With respect to the soil consistency interpretations, the N_{10} readings can be interpreted to give the respective granular and fine-grained soil consistencies as defined in the tables below:

Table 2.8: Consistency of granular soils from the DPL test (Nilsson, 2012)

Blows, N_{10}	**Consistency**	**Blows, N_{10}**	**Consistency**
Less than 1	Very Loose	7 - 83	Medium Dense
1 - 7	Loose	Over 83	Dense

Table 2.9: Consistency of fine soils from the DPL test (Nilsson, 2012)

Blows, N_{10}	**Consistency**	**Blows, N_{10}**	**Consistency**
Less than 3	Very Soft	13 - 22	Stiff
3 - 6	Soft	23 - 45	Very Stiff
6 - 12	Medium	Over 45	Hard

2.6.6 Soil bearing capacity evaluation based on Shear tests

2.6.6.1 Terzaghi's formula

Terzaghi's equation for computing bearing capacity either in general shear or local shear failure modes is shown below (Das, 2016; Das and Sobhan, 2018):

$$\mathrm{q_{all}} = \frac{\mathrm{q_{ult}}}{FOS} = \left[\mathrm{cN_c}S_c + 0.5\gamma_\mathrm{t}\mathrm{BN}_\gamma S_\gamma + \gamma_\mathrm{t}\mathrm{D_f}\mathrm{N_q}\right]/FOS \tag{2.26}$$

Where:

$q_{ult} = [cN_c S_c + 0.5\gamma_t B N_\gamma S_\gamma + \gamma_t D_f N_q]$; and $N_q = \left[\frac{a^2}{a\, cos^2\,(45 + \emptyset/2)}\right]$

$a = [e^{(0.75\pi - \emptyset/2)\ \tan\emptyset}]$; $N_c = [(N_q - 1)\cot\emptyset]$; $N_\gamma = \frac{\tan\emptyset}{2}\left(\frac{K_{pr}}{cos^2\,\emptyset} - 1\right)$

Where:

q_{all} = ultimate bearing capacity

B = width of the strip footing; L = length of the strip footing

γ_t = total unit weight of the soil; N_c, N_γ and N_q = bearing capacity factors

D_f = vertical distance from ground surface to bottom of the strip footing

c = cohesion of the soil; and FOS = Factor of Safety (Assumed as 3)

Table 2.10: Shape factors for Terzaghi's equation (Bowles, 1997)

For:	**Strip foundation**	**Round foundation**	**Square foundation**
$S_c =$	1.0	1.3	1.3
$S_\gamma =$	1.0	0.6	0.8

2.6.6.2 Meyerhof's formula

These Meyerhof's equations (1963) are for bearing capacity calculation:

$$q_{ult} = cN_cS_cd_c + q_oN_qS_qd_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma \text{ (Vertical Load)} \quad (2.27)$$

$$q_{ult} = cN_cd_ci_c + q_oN_qd_qi_q + 0.5\gamma BN_\gamma d_\gamma i_\gamma \text{ (Inclined Load)} \quad (2.28)$$

Where: $N_q = e^{(\pi\tan\emptyset)}\, tan^2\left(45 + \frac{\emptyset}{2}\right)$; $N_c = (N_q - 1)\cot\emptyset$; and

$N_\gamma = (N_q - 1)\tan(1.4\emptyset)$

2.6.6.3 Hansen's formula

These equations of Hansen (1970) are for bearing capacity calculation:

$$q_{ult} = cN_cS_cd_ci_cb_cg_c + q_oN_qS_qd_qi_qb_qg_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \quad (2.29)$$

$$q_{ult} = (\pi + 2)S_u(1 + S_{su} + d_{su} - i_{su} - b_{su} - g_{su}) + q_o \quad (2.30)$$

Where: $\mathrm{N_q} = e^{(\pi \tan \emptyset)}\, tan^2(45 + \emptyset/2) = \mathrm{Meyerhof's}$

$\mathrm{N_c} = \left(\mathrm{N_q} - 1\right) \cot \emptyset = \mathrm{Meyerhof's\ above;\ and}\ \mathrm{N_\gamma} = 1.5\left(\mathrm{N_q} - 1\right) \tan \emptyset$

2.6.6.4 Vesic's method

The Vesic (1973, 1975) procedure is essentially the same as Hansen's (1961) method with selective changes in N_γ, i_i, b_i, and g_i, but somewhat easier to use because Hansen's (1961) method uses the i terms in computing shape factors (S_i) whereas Vesic does not (Bowles, 1997; Das and Sobhan, 2018).

$$q_{ult} = cN_cS_cd_ci_cb_cg_c + q_oN_qS_qd_qi_qb_qg_q + 0.5\gamma BN_\gamma S_\gamma d_\gamma i_\gamma b_\gamma g_\gamma \qquad (2.31)$$

$$q_{ult} = (\pi + 2)S_u(1 + S_{su} + d_{su} - i_{su} - b_{su} - g_{su}) + q_o \qquad (2.32)$$

Where:

$\mathrm{N_q} = e^{(\pi \tan \emptyset)}\, tan^2(45 + \emptyset/2)$; and $\mathrm{N_c} = \left(\mathrm{N_q} - 1\right) \cot \emptyset = \mathrm{Meyerhof's}$

$\mathrm{N_\gamma} = 2\left(\mathrm{N_q} + 1\right) \tan \emptyset$

2.6.7 Effect of water table on bearing capacity calculations

In developing bearing-capacity equations, Terzaghi (1943), Meyerhof (1951 and 1963), Hansen (1970), and Vesic (1973, 1975) among other researchers, assumed that groundwater table was located at a depth much greater than the width (B) of the footing. However, if the groundwater table is close to the footing, the equation (2.33) below (Terzaghi's bearing capacity equation) or related equations for square, rectangular and circular footings. Other researchers' bearing capacity equations do require changes in the q and γ when calculating bearing capacities (Bowles, 1997; Das and Sobhan, 2018).

$$q_{ult} = q_c + q_q + q_\gamma = \left[c'N_c + qN_q + \frac{1}{2}\gamma BN_\gamma\right] \qquad (2.33)$$

With reference to Figure 2.5 below, three different cases of groundwater table locations with respect to the footing bottom are given in Figure 2.5 below.

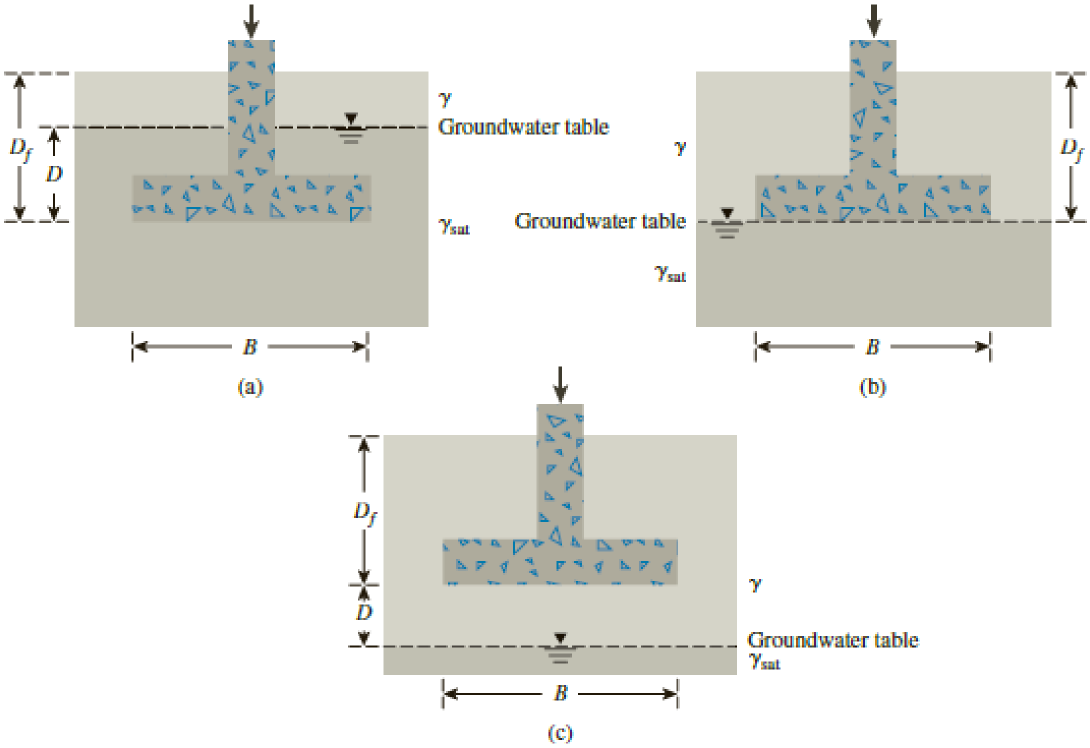


Figure 2.5: Effect of groundwater table on bearing capacity (Das and Sobhan, 2018)

a) Case 1 (Figure 2.5 (a))

If the groundwater table is located at a distance D above the bottom of footing, the magnitude of q is calculated as:

$$q = q_o = \gamma(D_f - D) + \gamma' D \qquad (2.34)$$

Where: $\gamma' = (\gamma_{sat} - \gamma_{wet})$ and replaces γ in the bearing-capacity equations.

b) Case 2 (Figure 2.5 (b))

If the groundwater table coincides with the bottom of footing, the magnitude of q equal γD_f, and γ is replaced by γ' in the bearing-capacity equations used.

$$q = \gamma \mathrm{D_f} \qquad (2.35)$$

c) Case 3 (Figure 2.5 (c))

When the groundwater table is at a depth D below the bottom of the footing, $q = \gamma D_f$. The magnitude of γ in the bearing capacity equations should be replaced by γ_{av} as shown below:

$$\gamma_{av} = \frac{1}{B}[\gamma D + \gamma'(B - D)] \text{ for } (D \leq B) \tag{2.36}$$

$$\gamma_{av} = \gamma \text{ for } (D > B) \tag{2.37}$$

CHAPTER THREE

METHODOLOGY

3.1 Introduction

This chapter presents the methodologies of the geotechnical investigation testing programme, prescriptive geotechnical and structural foundation designs, and the static foundation load test method used.

Due to the complex nature of the soil-foundation relationship and subsequent load-settlement response, a comprehensive geotechnical investigation was carried out to define the characteristics of the subsurface soil materials, ground water tables, estimate soil design parameters and the resistivity of the soils within the foundation sites that formed a basis for the prescriptive foundation design. This study was conducted on a total of four (4) foundation sites across the 400 kV and 132 kV overhead transmission lines of the Karuma Interconnection Project (KIP) in Uganda. The geotechnical investigation consisted of conventional sampling and laboratory testing as well as in-situ static foundation load testing. Conventional sampling methods included undisturbed samples obtained with a thin-walled Shelby tube sampler as well as disturbed samples using a standard split-spoon sampler. In-situ tests included the standard penetration (SPT) test, dynamic probing light (DPL) tests, and static load tests.

The laboratory tests were performed on the soil samples to determine particle size distributions, Atterberg limits, soil classifications, shear strength and consolidation characteristics.

3.2 Project location

The 400 kV Karuma-Kawanda Transmission Line was a 248.14 km long overhead powerline that traversed the districts of Wakiso, Luwero, Nakasongola, Masindi, and Kiryandongo in Uganda; while the 132 kV Karuma-Lira Transmission Line was located along the Karuma-Kamdini-Lira road.

The location map of the project area is shown in Appendix A.1.

3.3 The Site Geology

3.3.1 General Geology

The geology of the Karuma Interconnection Project (KIP) with respect to the quaternary era was covered by alluvium, swamps, and lacustrine deposit soils. It also had soils belonging to the Buruli-catena characterised by Ferrallitic soils, mainly red sandy-loams as presented in Appendix A.2.

The geology of Karuma-Lira Transmission Line was mainly underlain by the undifferentiated grabities and north granulites facie rocks (basement complex) as presented in Appendix A.2; whereas the geology for Karuma-Kawanda Transmission Line was underlain by gneissic granulitic complex rocks which were high grade metamorphics that included intermediate granulites and charnockites, quartz diorites, porphyroblastic and quartz-feldspathic types with the subsurface conditions dominated with silty clay soils, lateritisation and duricrust.

3.3.2 Local Geology

The local geology along Karuma-Lira Transmission Line mainly consisted of the Quaternary rock system (Alluvium deposits and Laterite), Neoarchaean rock system [A_3Umgb Metagabbro], Neoarchaean rock system [A_3Ugrdg Awela granodiorite gneiss (2649 ± 6 Ma)] and Neoarchaean rock system [A_3Ubttg Gulu banded TTG gneiss (2652 ± 8 Ma)], and Neoarchaean Amuru group (A_3Ubgn Banded gneiss and A_3UAbhgn Biotite-hornblende gneiss).

The Karuma-Kawanda Transmission Line's local geology (up to Masindi Port) mainly consisted of Quaternary rock system (QH1 Laterite and QHu Alluvium, swamp and lacustrine deposits), Neoproterozoic rock system [P_3MBsh Hoima mudstones, shales, slates and phyllites of Bunyoro group (765 Ma to 735 Ma and younger)], Mesoproterozoic rock system [P_2Ims Murchison mica schists, and P_2Ibif Lere banded iron formations of the Igisi group (~1.0 Ga)], Neoarchaean rock system [A_3do metadolerite, A_3Ugrdg Awela granodiorite gneiss [2649 ± 6 Ma (Mega-anum)] and A_3Upggn Porphyritic granite gneiss].

3.3.3 The Soils

The Karuma-Lira Transmission Line soils mainly consisted of Amuria-catena, Buruli-catena, Sesse series and undifferentiated alluvium, whereas the Karuma-Kawanda Transmission Line traversed areas underlain by gneissic granulitic complex rocks as well as the Buganda-Toro systems. The relevance of presenting the areas' soil catena was to show the geological parent materials from which the current soils were formed when compared to geotechnical soil profiles with respect to depth, soil moisture or acidity (Waugh, 2000; Schaetzl and Anderson, 2005) and geohydrology among a series of distinct but co-evolving soils in the same climate. Thus, Appendix A.2 presents the areas' soil catena under a generalised geology map of Uganda modified after Macdonald (1966), and Muwanga *et al*. (2001).

3.3.4 Seismicity

The proposed sites are located in Zone 2 of the Uganda Seismicity map (MoWT, 2010) as presented in Appendix A.3. Zone 2 implies a medium seismic risk level with a seismic zoning factor of Z = 0.8 for purposes of design (US 319, 2003; Delvaux *et al*., 2015), and a maximum peak ground acceleration (PGA) at the bedrock of 0.1g (where $g = 9.81 m/s^2$) as shown in Table 3.1:

Table 3.1: Uganda's seismic zones (Newmark and Hall, 1969; US 319, 2003)

Zones	**Seismic zoning factor (Z)**	**Bedrock acceleration (A)**	**Seismic Risk Level Interpretation**
1	1.0	0.1g - 0.23g	High seismic risk level
2	0.8	0.07g - 0.1g	Medium seismic risk level
3	0.7	0.05g - 0.07g	Low seismic risk level

Note: $g = 9.81 m/s^2$

3.3.5 Climatic Zones

The proposed project area was located in the demarcated rainfall Zone I (MoWT, 2010) consisting of Eastern and Northern Uganda (Adjumani, Gulu, Apac, Western Lira, and Eastern Masindi districts) that receives an average mean rainfall of 1340 mm as presented in the Appendix A.4. The seasonal rainfall distribution patterns over Uganda can be generalised into the four broad seasons (MoWT, 2010).

- Season 1: Generally dry period that lasts from December of the preceding year to the end of February.
- Season 2: The main rainy season throughout Uganda and referred to locally as the 'long rains' lasts from March to the end of May.
- Season 3: Dry except in parts of northern Uganda during June-August.
- Season 4: The second rainy period throughout the country and known locally as the 'short rains', lasts from September to the end of November.

These rainfall patterns affect ground water levels because of seasonal effects. Thus, the geotechnical engineer may provide some allowances depending on the time of year when the works were carried out or when it rained.

3.4 Parameters for geotechnical and structural designs

This section was used to establish parameters for geotechnical and structural designs of the foundations. This was through the understanding of the soil's geotechnical properties which is a prerequisite to conducting the foundation's prescriptive geotechnical and structural designs of load capacities and settlements. The insitu exploratory tests included test trial pits, borehole drilling, dynamic probing light (DPL) test, standard penetration test (SPT), and soil resistivity tests. The laboratory tests performed on the soil samples included among others, the natural moisture content, particle size distribution, Atterberg limits, pH value, sulphate and chloride content, specific gravity, consolidation testing and shear strength testing, and bulk density.

3.4.1 Test Trial Pits

a) Objective

The objective was to give the engineer a simple and direct access to the insitu subsurface soil strata with their associated depths so as to visually appraise the soil conditions and obtain samples for laboratory analysis.

b) Reference Literature

- BS 5930: 1999+A2: 2010
- BS 6031: 2009

c) Significance

- The test trial pit helped in the visual assessment and recording of the soil stratification, presence of ground water table, presence of hard rock, and obtaining other insitu information that were used in the creation of comprehensive test trial pit logs for the site locations.
- They provided the opportunity to conduct in-situ tests such as hand shear vane, and/or collection of samples for laboratory and chemical testing.

d) Apparatus

- Measuring tape or any survey machine
- Digging tools e.g. hoe, shovel, pickaxe, scoop, or backhoe excavator.
- Wooden pegs and strings or lime powder
- Sisal bags or canvas or tarpaulin
- Plastic moisture bags/Air-tight jar
- Minimum of three (3) workers

e) Procedure

i) The locations of the trial pits were marked at a distance of 1m from the survey mark stone so as not to disturb the final foundation's coordinate point.

ii) Using a manual procedure, the trial test pits were excavated to the dimensions of 1m x 0.5m x 3m (L x W x D) after being marked out by strings and pegs.

iii) The excavations were done layer by layer, and each stripped soil was stored on site separately from other excavated material.

iv) Jar samples were taken of the excavated soils at every metre depth or, if more frequent, at every stratum change.

v) The trial pits were excavated to a maximal depth of 3-5m, or until the excavated sides became unstable, and/or bedrock was reached.

vi) Following water strike, the flow and level were recorded over a 20-minute period, and ground water samples taken.

vii) When the trial pit was completed, the engineer and the technician recorded and photographed all details regarding the trial pit.

viii) The trial pit was backfilled in reverse order, in which it was excavated and compacted to reduce later settlement; however, in a few cases where at the end of the working day there was an exploratory hole not backfilled, it was securely covered and barricaded so as to prevent human or animal access.

f) Expected Results

- Trial pit logs detailing the depth and brief nature of the strata encountered.
- Description of the various strata encountered and/or removed.
- The presence or absence of water in the test pit and the depth encountered.
- Coordinate of the location and terrain description.

Figure 3.1: Typical excavated trial pits

3.4.2 Dynamic Probing Light

a) Objective

The objective of the Dynamic Probing Light (DPL) test was to test the thickness of the soil layers, the control of soil consistency, and the determination of strength and deformation parameters to a depth of 8-12m if the ground was not too dense.

b) Reference Literature

- BS EN ISO 22476-2: 2005 + A1: 2011
- DIN 4094: 1990

c) Significance

- DPL provided a measure of the material's in-situ resistance to penetration, the strength of insitu soil, the thickness and location of underlying strata.
- Test results could be correlated to California Bearing Ratios (CBR), in-situ density, resilient modulus, and bearing capacity.

d) Apparatus

The Lightweight Dynamic Cone Penetrometer equipment consists of:

- 1 No. manual DPL testing apparatus with 10 kg drop-weight.
- 1 No. Jack-type tube puller.
- 6 No. SPT/DPT sounding tube (22 x 1000 mm) with 10 cm markings
- 6 No. Threaded nipple studs of M16 x 50 for 22 mm sounding tubes.
- 1 No. Conical sounding tip of tempered steel - 5 cm^2, M16 connection.
- 1 No. Conical sounding tip of tempered steel - 10 cm^2, M16 connection.
- 20 No. Conical sounding tip - 10 cm^2 - single use type.
- 2 No. Open-ended spanner 19 mm for connecting the 22 mm tubes.

- 1 No. Allen key (M8) for threaded nipples.
- 1 No. Sturdy wooden transport case for transport and storage.
- Total weight of the complete set is approximately 60 kg.

e) Procedure

i) The hammer mass of about 10 kg was dropped freely under gravity through a 500mm height-fall to drive the cone of 0.001 m^2 cross-sectional area.

ii) The number of blows per 100 mm penetration into the ground (N_{10}) were then consecutively read and recorded throughout the entire 1m length of the rod, and the N_{10} values were interpreted to give the unit point resistance, r_d and the dynamic point resistance, q_d.

iii) When the first rod reached a depth of 800 mm into the ground, the next rod was then securely placed, and the process was repeated until refusal. Thereafter, the rods were withdrawn using an extrusion lever.

iv) Whenever possible, the dynamic penetrometer testing was performed at rates of 15 to 30 strokes per minute, without pausing.

v) The number of strokes were calculated after every 10 cm of penetration depth. For individual tests in very soft or loose soils where the penetration depth of 10 cm was reached after 1 to 3 strokes, or for very consistent or dense soils where the penetration depth of 10 cm was not achievable, penetration depth at certain number of strokes (< 5 or > 50) were measured.

f) Expected Results

- The number of blows per 10cm (100mm) penetration (N_{10}).
- The penetration rate in metres per blow (e).

- The unit point resistance (r_d) and dynamic point resistance (q_d) in Mpa.
- Graphical depictions of depth vs N_{10}, depth vs r_d, and depth vs q_d.

Figure 3.2: Dynamic Probing Light (DPL) test in the field

3.4.3 Standard Penetration Test

a) Objective

The objective of the Standard Penetration Test (SPT) was to determine the relative density and bearing capacity of granular sandy soils and/or comparative strengths of underlying soil strata based on established penetration N-values, that could be empirically related to many engineering properties.

b) Reference Literature

- BS EN ISO 22476-3: 2005
- ASTM D1586-99: 1999

c) Significance

- The SPT data was used to estimate both the strength and stiffness parameters for bearing capacity and settlement analysis of foundations.

d) Apparatus

Standard Penetration Test (SPT) tools consisted of the following:

- Drilling Rig
- Standard Split Spoon Sampler, and Shelby tube sampler
- Wax and sample-sealing material
- Chalk, metre-ruler, and Soil sampling jars
- Drop Hammer weighing 63.5kg
- Driving head (anvil).
- Guiding rod, Tripod assembly, and Extension rods.

e) Procedure

i) The Standard split-barrel Sampler (Split-spoon) was attached to the bottom of the drilling rod, while the top of the drilling rod attached by anvil was used to transfer the hammer load to the drilling rod. The anvil was connected to a guide rod passing through the drop hammer.

ii) The tripod was erected so that each leg formed an angle of 120° with respect to the other and equidistant from the centre mark. The pulley was hooked-up to the tripod with a rope passing over it, and one end of the rope connected with the drop hammer to lift it up.

iii) A rectangular trench was excavated to the required foundation depth (below which the soil's bearing capacity was required) at the centre mark.

iv) Gradually pulled the other end of the rope (manually or by some mechanical arrangements) to erect the sampler. Made sure that the sampler assembly was vertically erected at the centre mark of the testing spot.

v) The rope was pulled slowly to lift the drop hammer to the full height of the guide rod (76 cm approximately) and then the rope was suddenly released to provide free fall to the hammer repeatedly to drive the Standard split-barrel Sampler (Split-spoon) 18" (~450mm) into the soil.

vi) After driving the sampler 18" (~450mm) into the soil, the number of blows required to penetrate each of the three 6" (~150mm) increments were counted. The Standard Penetration Resistance value (N-value) was then considered as the number of blows required to penetrate the last 12" (~300mm).

vii) After the blow counts had been obtained, the split-spoon sampler was removed and opened to obtain a disturbed sample for subsequent testing.

viii) The specific weight of the soil was then determined on the spot of the boring log to obtain the effective overburden pressure.

f) Expected Results

- The number of SPT depth, number of seating and driving blows, measured SPT N-values, soil consistency description.
- The overburden correction factor (C_N), hammer factor (η_1), rod length factor (η_2), sample factor (η_3), borehole diameter factor (η_4), overall correction factor (C_{ER}) and corrected SPT N-values.
- The ultimate (q_{ult}) and/or allowable bearing capacity (q_{all}) in kPa.

3.4.4 Borehole drilling

a) Objective

The objective of borehole drilling was to extract core samples as part of deep subsurface exploration aimed at obtaining disturbed and undisturbed samples for visual examination and laboratory testing.

b) Reference Literature

- BS 5930: 1999

c) Significance

- Borehole drilling was more efficient in obtaining samples from significant depths for visual inspections and testing.
- With borehole drilling, overlying competent materials could be penetrated to assess the nature of underlying less competent strata to help assess the in-situ properties of the ground.

d) Apparatus

- Rotary drilling machine (GY-50 Conventional drilling rig)
- Core cutting barrels, Segmented rod, drilling rods and cutting bits.
- Machine stands/supports, Pulley systems, fork and chain spanners, casings
- Horse pipes, spade, Pickaxe, Hoe, Twist bars, spirit level/plumb bob.
- Jerrycans, metallic/halved plastic drum, and Sample Boxes

e) Procedure

i) The borehole centre marking and position coordinates were marked before setting and positioning the rotary drilling machine.

ii) After the rotary machine had been assembled, the drill rig was positioned on top of the marked point, and the drill rod with bits placed over the mark.

iii) To ensure verticality checks, a spirit level and/or plumb bob was used.

iv) The drilling works were commenced with the installation of the first 4m casing to minimise the collapse of the top 4m soil strata.

v) SPT Tests were conducted in all the boreholes at intervals of 1.5m.

vi) The soil samples and strata profiles were recovered and recorded, and the recovered samples stored in the clearly labelled sample boxes.

vii) The ground water table was also monitored and recorded.

viii) The diameter of drilled holes varied from 150-50mm as depth increased.

ix) The undisturbed samples from the bored pits were collected and stored in sealed tubes and conveyed with extreme care to avoid vibration and impact-blows that could cause disturbance in samples.

x) The disturbed samples were packed in plastic bags after being recovered to avoid changes in moisture contents and contamination from drill-slimes.

f) Expected Results

- Boring log soil-profile with associated soil descriptions, SPT N-values and investigation core photos.

Figure 3.3: Laboratory team conducting borehole drilling

3.4.5 Natural Moisture Content

a) Objective

The objective of the test was to determine the amount of water present in the soil expressed as a percentage of the mass of dry soil.

b) Reference Literature

- BS 1377: Part 2: 1990
- ASTM D2216-98: 1998

c) Significance

- The moisture content was one of the most important index properties used in establishing a correlation between soil behaviour and its index properties.
- The water content of the material was used in expressing the phase relationships of air, water, and solids in the given volume of material.
- In fine-grained (cohesive) soils, the consistency of a given soil type depended on its water content. The water content of a soil, along with its liquid and plastic limits as determined by Test Method D 4318, was used to express its relative consistency or liquidity index.

d) Apparatus

- Drying oven with temperature of 105°C to 110°C
- Balance readable to 0.1g
- Metal container, and Desiccator

e) Procedure

i) The container was cleaned, dried, and weighed to the nearest 0.1g (m_1).

ii) A representative sample was crumbled and loosely placed in the container: For example, the minimal sample weight used was 30g for fine-grained soils, 300g for medium-grained soils, and 3kg for coarse-grained soils.

iii) The container with sample altogether, were immediately weighed (m_2) and placed in the oven to dry at 105ºC for a minimum of 12 hours.

iv) After drying, the container and its contents altogether, were weighed (m_3).

f) Expected Results

- The Moisture Content of the soil specimen, W, calculated as a percentage of the dry soil mass to the nearest 0.1%, from the equation:

$$W = \left(\frac{m_2 - m_3}{m_3 - m_1}\right) x\ 100\% \qquad (3.5)$$

Where:

m_1 = mass of container (in g)

m_2 = mass of container and wet soil (in g)

m_3 = mass of the container and dry soil (in g)

3.4.6 Particle Size Distribution (Sieve Analysis)

a) Objective

The objective of this test method was to separate particles into different grain size ranges and to quantitatively determine the mass of particles in each range. This test method used a square opening sieve criterion in determining the gradation of soil between the 3-inch size (75mm) and No. 200 (75µm) sieves.

b) Reference Literature

- BS 1377: Part 2, Sub-clause 9.2: 1990
- ASTM D6913/D6913M - 17: 2017; and ASTM D2487-17: 2017

c) Significance

- The soil gradation was used for classification as per ASTM D2487-17 (2017).
- The gradation (particle-size distribution) curve was used to calculate the coefficient of uniformity and the coefficient of curvature.
- Selection and acceptance of fill materials were based on gradation, for example, foundation backfills, like earthen dams have gradation requirements.
- The gradation of a soil was an indicator of its engineering properties. Hydraulic conductivity, compressibility, and shear strength are related to the gradation of the soil although the engineering behaviour is dependent upon many factors such as effective stress, plasticity, and geologic origins.

d) Apparatus

- Mechanical sieve shaker, sieves (standard sieve set, washing sieve, No. 200 (75-µm), and designated separating sieve), and sieving containers (specimen containers, collection device and cumulative mass container)
- Washing sink with spray nozzle, balances, drying oven, sieve brushes
- Miscellaneous items such as wash bottle, spatula, and stirring rod
- Riffle box, quartering accessories, mortar, and rubber-covered pestle

e) Procedure

i) The weight of each sieve together with the bottom pan to be used in the analysis were written down, and the weight of the dry soil sample were recorded.

ii) All the sieves were cleaned and assembled in the ascending order of sieve numbers (75 mm sieve at top and 75 µm sieve at bottom). The pan was then placed below the 75 μm sieve, and the soil sample carefully poured into the top sieve and the cap placed over it.

iii) The sieve stack was placed in the shaker and shaken for 10 minutes.

iv) Thereafter, the stack was removed from the shaker and the weight of each sieve with its retained soil and also the bottom pan with its retained fine soil were carefully weighed and recorded.

v) The mass of soil retained on each sieve was obtained by subtracting the weight of the empty sieve from the mass of the sieve + retained soil and recorded this mass as the weight retained on the data sheet. The sum of these retained masses approximated the initial mass of the soil sample.

vi) The percentage-retained on each sieve was calculated by dividing the weight retained on each sieve by the original sample mass.

vii) The percentage-passing was calculated by starting at 100% and subtracting the percentage-retained on each sieve in a cumulative procedure.

viii) A semi-logarithmic plot of grain size versus percentage-passing was made.

f) Expected Results

- Prepared calculation sheet for soil particle percentage passing detailing the weight retained on each sieve, percentage retained, and cumulative percentage retained on each sieve, and percentage passing each sieve.
- A semi-logarithmic plot of grain size against the percentage passing.

3.4.7 Liquid Limit Tests (Cone Penetrations Method)

a) Objective

The objective was to determine the Liquid Limit (LL), which is the empirically established moisture content where a soil passes from the liquid to plastic state.

b) Reason for using Cone Penetration Method

- It was easier to perform in laboratory, and the cone penetrometer results don't depend on the operator’s judgement, thus, making the results very reliable.
- The results could be used to estimate the undrained shear strength of soils.

c) Reference Literature

- BS 1377: Part 2, Sub-clause 4: 1990; and ASTM D4318 - 17: 2017

d) Significance

- Liquid limit was important for classifying fine-grained or cohesive soils.
- Liquid Limit gave information on the soil’s insitu state of consistency.
- Liquid Limit was used to predict the consolidation properties of soils while calculating allowable bearing capacity and settlement of foundations.
- Liquid Limit value of a soil was used to calculate the activity of clays and toughness index of the soil.

e) Apparatus

- Test sieves (425 μm), air-tight container, and moisture content apparatus
- A flat glass plate, and a metal straight edge
- Two palette knives or spatulas, penetrometer, and stopwatch
- 80g and 35 mm long, polished stainless-steel cone of a 30° angle.
- Metal cup (Ø55 mm and 40 mm deep) with the rim parallel to the flat base
- An evaporating dish or a damp cloth, and a wash bottle with clean water

f) Procedure

i) A sample of the soil of sufficient size was taken to give a test specimen weighing about 400g which passes the 425 mm sieve.

ii) The soil was then transferred to a glass plate, water added, and thoroughly mixed with two palette knives to form a thick homogeneous paste.

iii) The paste was later placed in an airtight container and allowed to stand for 16-24 hours to enable the water to permeate through the soil.

iv) The 400g soil sample was taken and placed on a glass plate, and the paste mixed for at least 10 minutes using the two palette knives.

v) A portion of the mixed soil was pushed into the cup with a palette knife, taking care not to trap air, and gently tapping the cup against a firm surface. Any excess soil was cut-off with the straight edge to give a smooth level.

vi) With the penetration cone locked in the raised position, the cone was raised so as to just touch the surface of the soil. When the cone was in the correct position, the cup's slight movement was used to mark the soil surface. The dial gauge was lowered to make contact with the cone shaft and the reading of the dial gauge recorded to the nearest 0.1 mm.

vii) The cone was released for a period of 5 ± 1 seconds, the cone locked in position, and dial gauge lowered to make contact with the cone shaft so as to record the read to the nearest 0.1 mm. The difference between the readings was recorded as the cone penetration value.

viii) The cone was then lifted-out and carefully cleaned.

ix) A little more wet soil was added to the cup and the process repeated. However, when the difference between the first and second penetration readings was less than 0.5 mm, the average of the two penetrations was recorded. When the second penetration was more than 0.5 mm and less than 1 mm or different from the first, a third test was carried out. If the overall range was then not more than 1 mm, the average of the three penetrations was recorded. In rare cases where the overall range was more than 1 mm, the soil was removed from the cup, remixed and the test repeated until consistent results are obtained.

x) A moisture content sample of 20g was taken from the area penetrated by the cone and its moisture content determined.

xi) The penetration test was repeated at least three more times using the same sample of soil, to which further increments of water had been added. The amount of water added was such that a range of penetration values of approximately 15 mm to 25 mm was covered by the four test-runs.

xii) The cup was washed and dried each time the soil was removed from it.

g) Expected Results

- Calculated moisture content of each specimen using equation 3.6:

$$w = \left(\frac{m_1 - m_3}{m_3 - m_1}\right) x\ 100\% \qquad (3.6)$$

Where:

m_1 = mass of container (in g); m_2 = mass of container and wet soil (in g)

m_3 = mass of container and dry soil (in g)

- The plot of the relationship between the moisture content and cone penetration with the moisture content as the abscissae and the cone penetration as the ordinates, both on linear scales.
- Drawing of the best straight line of fit across the points.
- The Liquid Limit (w_L) determination as the moisture content of the soil corresponding to the cone penetration value of 20mm.

3.4.8 Liquid Limit Tests (Casagrande Method)

a) Objective

The objective was to determine the Liquid Limit (LL), which is the empirically established moisture content where a soil passes from the liquid to plastic state.

b) Reason for using Casagrande Method

- It was an alternative to the cone penetration method.
- The results could be used to estimate the undrained shear strength of soils.

c) Reference Literature

- BS 1377: Part 2, Sub-clause 4: 1990
- ASTM D4318 - 17: 2017

d) Significance

- Liquid limit was used in the classification of fine-grained/cohesive soils.
- Liquid Limit gave information on the soil's insitu state of consistency.
- Liquid Limit was used to predict the consolidation properties of soils while calculating allowable bearing capacity and settlement of foundations.
- Liquid Limit was used to calculate the activity of clays and toughness index.

e) Apparatus

- Casagrande's liquid limit device
- Grooving tools of both standard and ASTM types
- Oven, evaporating dish or glass sheet, and spatula
- 425 microns sieve
- Weighing balance, and Wash bottle

f) Procedure

i) The drop of the cup of the liquid limit device was adjusted by releasing the two screws at the top and by using the grooving tool handle. The drop of 1 cm was made at the point of contact on the base, and the screw tightened.

ii) 120g of the air-dried soil sample passing 425 microns sieve was taken, and the sample thoroughly mixed with distilled water a glass plate for about 15 to 30 min, to form a uniform paste mix.

iii) The mix was kept under humid conditions to obtain uniform moisture distributions for a sufficient maturing period of up to 24 hours.

iv) A portion of the matured paste was taken and thoroughly remixed, and placed in the cup of the device by a spatula and levelled with a straight edge to have a minimum depth of 1 cm soil at the point of the maximum thickness. Any excess soil was transferred to the evaporating dish.

v) A groove was cut in the sample in the cup using the appropriate tool, and a groove was drawn through the paste in the cup along the symmetrical axis, through the cup centre line.

vi) The handle of the device was turned at a rate of 2 revolutions per second, and the number of blows counted until the two halves of the soil specimen came into contact at the bottom of the groove along a distance of 12mm due to flow and not by sliding.

vii) A representative soil sample was collected by moving spatula width-wise from each edge of the soil-cake at right angles to the groove.

viii) The remaining soil was removed from the cup and mixed with the soil left in evaporating dish.

ix) The water content of the mix in the evaporating dish was changed either by adding more water if the water content was to be increased or by kneading the soil, if the water content was to be decreased.

x) The steps were repeated so as to determine the number of blows (N) and the water content in each case.

g) Expected Results

- The flow curve of *log N* against *w* drawn so as to determine the liquid limit corresponding to N = 25 blows.

3.4.9 Plastic Limit and Plasticity Index Tests

a) Objective

The objective was to determine the Plastic Limit (PL), which is the moisture content where the thread breaks apart at a 3.2 mm diameter. A soil is non-plastic if the thread cannot be rolled down to 3.2 mm at any moisture possible.

b) Reference Literature

- BS 1377: Part 2, Sub-clause 5: 1990
- ASTM D4318 - 17: 2017

c) Significance

- Plastic Limit (PL) was used together with the Liquid Limit to determine the Plasticity Index which when plotted against the Liquid Limit on the plasticity chart provided a means of classifying cohesive soils.
- The PL test was performed as a continuance of the LL test, and material for the test could conveniently be prepared as part of the Liquid Limit test.

- A wide variety of soil engineering properties have been correlated to the LL and PL values, and these limits were used to classify the fine-grained soils according to the Unified Soil Classification system or AASHTO system.

d) Apparatus

- Two flat glass plates, one for mixing soil and one for rolling threads
- Two palette knives or spatulas, and Clean water
- Apparatus for moisture content determination
- A length of rod, 3 mm in diameter and 100 mm long

e) Procedure

i) A 40g soil paste was taken and placed on a glass plate, and the soil dried partially until it became plastic enough to be shaped into a ball.

ii) The soil ball was moulded between fingers and rolled between the palms of both hands until the heat of the hands had sufficiently dried the soil for slight cracks to appear on its surface.

iii) The sample was divided into two portions of about 20g each, and separate determinations carried out on each portion. Then each of the two portions divided into four parts.

iv) The soil was moulded in between the fingers to equalise the moisture distribution, and then the soil formed into a thread of about 6 mm diameter between the first finger and thumb of each hand.

v) The thread was rolled between the fingers, from finger-tip to the second joint of one hand and the surface of the glass plate to reduce the thread diameter to 3 mm in about 5 to 10 complete forward and back movements.

vi) The soil was picked up, moulded between the fingers to dry it further, and formed into a thread and rolled out again as specified above.

vii) The procedure was repeated until the thread sheared in both longitudinal and transverse directions when rolled to 3 mm diameter. The first crumbling point was considered as the Plastic Limit.

viii) All the pieces of crumbled soil threads were gathered and transferred to a suitable container for moisture content determination.

ix) The rolling procedure was repeated on the other three sub-sample portions.

x) The rolling procedure was repeated on the second sub-sample so as to achieve two separate moisture content determinations. However, the whole test was repeated when the two results differed by more than 0.5%.

xi) The average of the two moisture content values was calculated, and the value expressed to the nearest whole number as the Plastic Limit (W_P).

xii) The Plasticity Index (I_P) was calculated as the difference between the Liquid Limit (WL) and Plastic Limit (W_P), as $I_P = W_L - W_P$

f) Expected Results

- The records for mass of empty container (m_1), mass of container and wet soil (m_2), and the mass of container and dry soil (m_3).
- The calculations of mass of water ($m_2 - m_3$), mass of dry soil ($m_3 - m_1$), and moisture content [$\left(\frac{m_2-m_3}{m_3-m_1}\right) x\ 100\%$].

3.4.10 Linear Shrinkage Limit Tests

a) Objective

The objective was to determine the Linear Shrinkage (LS) value so as to quantify the amount of shrinkage likely to be experienced by clayey material.

b) Reference Literature

- BS 1377: Part 2, Sub-clause 6.5: 1990
- ASTM D4318 - 17: 2017

c) Significance

- The Linear Shrinkage was considered a more reliable indicator than the Plasticity Index for materials with very low plasticity (that is, PI ≤ 6 %).
- Linear Shrinkage test offered a convenient way of confirming that the test results for the Plasticity Index were reasonable because most soil types exhibit a relationship between Plasticity Index and Linear Shrinkage.

d) Apparatus

- Palette knives or spatulas, mould, petroleum jelly, and distilled water.
- Drying oven, and Graduated ruler or Vernier callipers.

e) Procedure

i) A 150g sample from thoroughly mixed portion of bulk material passing 425 microns sieve was prepared.

ii) The mould was thoroughly cleaned, and a thin film of grease applied to its inner walls, and the soil sample thoroughly mixed with distilled water using palette knives until the mass became homogeneous.

iii) The thoroughly mixed soil-water paste was placed in the mould to be slightly above the sides of the mould.

iv) Then the mould with the soil paste dried in the oven at a maintained temperature of about 105°C to 110°C.

v) After complete drying, the mould and soil were cooled and the mean length of the soil bar (L_D) measured.

vi) The linear shrinkage (L_S) of the soil was calculated as a percentage of the original length of the specimen (L_0) from the following formula:

$$L_S = \left[1 - \left(\frac{L_D}{L_0}\right)\right] \text{ x } 100\% \tag{3.7}$$

f) Expected Results

- The recorded lengths of L_0 and L_D, and the calculated value of L_S.

3.4.11 pH Test

a) Objective

The objective of the pH test was to determine the degree of acidity (values less than 7) or alkalinity (values greater than 7) and to supplement the soil resistivity measurements and thereby identify conditions under which the corrosion of metals in soil may be accentuated.

b) Reference Literature

- BS 1377: Part 3: 1990
- ASTM G51 - 18: 2018

c) Significance

- pH value was an indicator of the corrosivity of a soil environment.

- The pH of the soil was a useful variable in determining the solubility of soil minerals and the mobility of ions in the soil.

d) Apparatus

- pH meter machine suitable for laboratory testing.
- Containers made of glass or wax coating, with moisture proof covers.
- pH buffer solutions of pH 4.0, 7.0 and 10.0 or those recommended by the pH meter manufacturer for meter standardisation.
- Distilled water and wash bottle.
- Thermometer readable to 0.1°C.
- 2.36 mm sieve.
- Balance, with sufficient capacity and readable to 0.1% of the sample mass, or better, conforming to the requirements of AASHTO M 231.
- Oven capable of maintaining a temperature of 60°C.
- Glass stirring rod.

e) Procedure

i) The received soils were oven dried at controlled temperature conditions not exceeding 60°C.

ii) A sufficient amount of the sample was quartered to yield approximately 100g of material taking care not to crush rock particles or naturally occurring grains and screened through a 2.36 mm sieve.

iii) Only natural material passing the 2.36 mm sieve was used for the test.

iv) 30.0 ± 0.1g sample of the prepared soil was placed into the test container, and an equal mass of distilled water placed inside the soil sample.

v) The mixture was stirred to obtain a slurry and covered.

vi) The sample was allowed to stand for a minimum of 1 hour, whilst stirring every 10 to 15 minutes.

vii) The pH meter was standardised as per the manufacturer's instructions and using the 7.0 pH buffer standard solution.

viii) The sample was stirred with a glass rod immediately prior to placing the pH meter electrode into the sample.

ix) The electrode was then placed in the soil-slurry sample and remained immersed in it for a sufficient time for the meter to stabilise.

x) The pH of the sample was then read from the pH meter machine and recorded to the nearest tenth of a whole number.

xi) The pH meter electrode was then cleaned by washing with distilled water and stored in accordance with the manufacturer's instructions.

f) Expected Results

- The pH value of soil suspension or ground water to the nearest 0.1 pH unit.

3.4.12 Sulphate and Chloride Content Test

a) Objective

The objective of the test was to sequentially determine the chloride and/or sulphate ions in water or soil by suppressed ion chromatography.

b) Reference Literature

- BS 1377: Part 3: 1990
- ASTM D4327 - 11: 2011

c) Significance

- The tests provided both qualitative and quantitative determination of anions such as carboxylic acids, Cl^{-} and SO_4^{2-}in milligram per litre (mL) range.

d) Apparatus

- Flasks, class A volumetric of 100 mL, 500 mL, 1000 mL capacities.
- Ion chromatograph, with an auto sampler.
- Pipettes, class A volumetric of 1 mL, 10 mL, 25 mL, 50 mL capacities.
- Vials (5 mL capacity) and caps for auto sampler.
- Certified anion standard reference solution, containing 100 parts per million (ppm) chloride and sulphate.
- Deionised or distilled water, meeting ASTM D1193, Type II requirements.
- Sodium bicarbonate eluent concentrate, for ion chromatograph of 8.0 mM sodium carbonate and 1.0 mM sodium bicarbonate after 100× dilution.

e) Procedure

i) 6 - 8 standards were prepared using the anion standard solution ranging from 0.1-100 ppm, after ensuring that the standards were well mixed.

ii) The ion chromatograph was set up and the standards run; and a calibration curve for Cl^{-} and SO_4^{2-} ions created using the results.

iii) For Chloride and Sulphate contents, 50 mL of the filtered sample was pipetted into a 500 mL volumetric flask and diluted to the mark, and the dilution well-shaken into a homogenous mixture.

iv) The manufacturer’s instructions were followed on how to start the ion chromatograph’s pump and electronic systems, by pumping the eluent through the column and detector until it attained a stable baseline.

v) Samples were poured into properly labelled sample vials, and one prepared standard and one de-ionized water blank were run after every 4-5 samples to check the accuracy of the chromatograph.

vi) The samples were run through the ion chromatograph to determine the concentration of the chloride and sulphate ions in conformity to the manufacturer’s recommendation for ion chromatograph operation.

vii) The Cl^- and SO_4^{2-} contents in ppm (parts per million) were obtained as determined by the ion chromatograph; and the conversion factors used to convert chloride and sulphate content in the original sample to ppm.

f) Expected Results

- Calculated dilution factor as per the equation:

$$\mathrm{D_F} = \frac{\mathrm{V_d}}{\mathrm{V_p}} \tag{3.8}$$

Where:

$\mathrm{D_F}$ = the difusion factor

$\mathrm{V_d}$ = the volume of the flask used for dilution (in mL)

$\mathrm{V_p}$ = the volume of the pipette used to make the dilution (in mL)

- Calculated concentration of standard solution used for calibration:

$$\mathrm{S_c} = \frac{\mathrm{C_s}}{\mathrm{D_F}} \tag{3.9}$$

Where:

$\mathrm{S_c}$ = standard solution concentration; and $\mathrm{D_F}$ = the diffusion factor used

$\mathrm{C_s}$ = concentration of the ion in certified reference solution (ppm)

- Calculated concentration of chloride ions in the original soil sample:

$$Cl = \frac{\mathrm{R\ x\ D_F\ x\ F}_l}{\mathrm{W}} \tag{3.10}$$

Where:

$Cl =$ concentration of chloride ions in the original soil sample (ppm)

$R =$ concentration of chloride ions in the sample run through the ion chromatograph (ppm)

$\mathrm{D_F} =$ difusion factor

$\mathrm{F}_l =$ volume of the flask containing undiluted sample (mL)

$W =$ weight of the original soil sample (g)

- Calculated concentration of Sulphate ions in the original soil sample:

$$SO_4 = \frac{\mathrm{R\ x\ D_F\ x\ F}_l}{\mathrm{W}} \tag{3.11}$$

Where:

$SO_4 =$ concentration of sulphate ions in the original soil sample (ppm)

$R =$ concentration of chloride ions in the sample run through the ion chromatograph (ppm)

$\mathrm{D_F} =$ difusion factor

$\mathrm{F}_l =$ volume of the flask containing undiluted sample (mL)

$W =$ weight of the original soil sample (g)

3.4.13 Bulk Density and Unit Weight Test

a) Objective

The objective of the test was to determine the bulk density of the soils, from which the unit weight of the soil may easily be derived. The unit weight of a soil is an essential parameter in most geotechnical engineering analyses, e.g. stability of slopes, consolidation settlement, earth pressure and bearing capacity analyses.

b) Reference Literature

- BS 1377: Part 2: 1990
- ASTM D7263 - 09: 2018

c) Significance

- Bulk density test could be used to convert the water fraction of the soil from a mass basis to a volume basis and vice-versa. When the specific gravity is known, dry density can be used to calculate porosity and void ratio.
- Dry density was also useful for determining the degree of soil compaction.
- Unit weights of remoulded specimens were used to evaluate the degree of backfill compaction when the dry density values were used in conjunction with compaction curve values.

d) Apparatus

- Balance readable to 1g; and Sample extruder
- Apparatus and equipment for moisture content determination
- Drying Oven; and Thermometer
- Apparatus for handling hot containers

e) Procedure

i) After the registration of the cylindrical tube sample, the lids were unscrewed, and the wax removed.

ii) The cylinder with the sample inside were weighed, and the mass recorded to the nearest 1g (m_T).

iii) The length of the sample in the cylinder was determined by measuring the length of the cylinder (l_1) and the depths from both ends of the cylinder (l_2 and l_3). The sample's average depths were recorded.

iv) The sample was extruded by following the relevant procedures for further tests to be carried out on the sample.

v) Thereafter, the mass (m_c) of the cleaned and dried cylinder was weighed.

vi) The moisture content was then determined on three different sample specimens, in order to achieve the average condition of the sample.

f) Expected Results

- Calculated Bulk density of the sample, ρ (in kg/m^3) expressed to the nearest 1 kg/m^3:

  $$\rho = \frac{M}{\mathrm{V}} = \left[\frac{(m_T - m_c)}{V}\right] x\ 100\% \qquad (3.12)$$

  Where:

  $\mathrm{m_T}$ = the mass of the cylinder and sample (in g)

  $\mathrm{m_c}$ = the mass of the empty cylinder (in g)

  V = the volume of the sample (in cm^3)

- Calculated Unit weight of the sample, γ (in kN/m^3) expressed to the nearest 0.01 kN/m^3 as $\gamma = \frac{W}{\mathrm{V}} = \frac{Mg}{V} = \rho\ x\ g = \rho\ x\ 9.81\ x\ 10^{-3}$

  Where:

  g = the acceleration due to gravity (= 9.81 m/s^2)

- Calculated dry density, void ratio and degree of saturation (where possible)

3.4.14 Specific Gravity (Particle Density Tests)

a) Objective

The objective of the test was to determine the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature.

b) Reference Literature

- BS 1377: Part 2, Sub-clause 8: 1990
- ASTM D854-14: 2014

c) Significance

- The specific gravity of a soil solids was used in calculating the phase relationships of soils, such as void ratio and degree of saturation.
- The specific gravity of the soil solids was used to calculate the density of the soil solids by multiplying its specific gravity by the density of water.

d) Apparatus

- Pycnometer, weigh-balance, Drying Oven, Thermometer and Desiccator
- Entrapped air removal apparatus such as a Hot Plate or Bunsen Burner
- Insulated container, and Funnel
- Pycnometer Filling Tube with Lateral Vents (optional)
- Sieve - No. 4 (4.75mm), and Blender
- Miscellaneous Equipment, such as a computer or calculator (optional), specimen dishes, and insulated gloves

e) Procedure

i) The density bottle was washed clean with water, dried, allowed to drain, and the empty bottle with its stopper weighed (W_1).

ii) 10-20g of oven-soil sample that had been cooled in a desiccator was taken, transferred to the bottle, and after the bottle and soil weighed (W_2).

iii) 10ml of distilled water was put in the bottle to allow the soil to soak completely for about 2 hours.

iv) Again, the bottle was completely filled with distilled water, and the stopper placed on top, and the bottle kept under constant temperature ($T_x{}^oC$).

v) The bottle was taken, wiped clean and dried, and the weight of the bottle and its contents (W_3) determined.

vi) The bottle was emptied and thoroughly cleaned and filled only with distilled water and weighed (W_4) at temperature ($T_i{}^oC$).

vii) The same process was repeated for 2 to 3 times, to take its average reading.

f) Expected Results

- The Specific gravity of soil calculated from the equation:

$$G_s = \frac{\text{Density of water at } 27^{o}\text{C}}{\text{Weight of water of equal volume}} \qquad (3.13a)$$

$$\Rightarrow G_s = \frac{(W_2-W_1)}{(W_4-W_1)-(W_3-W_2)} = \frac{(W_2-W_1)}{(W_2-W_1)-(W_3-W_4)} \qquad (3.13b)$$

Where:

W_1 = weight of empty bottle with its stopper only

W_2 = weight of bottle and dry soil sample only

W_3 = weight of bottle and dry sample and water only

W_4 = weight of bottle and water only

- Unless otherwise specified, the specific gravity values reported shall be based on water at 27°C, implying the specific gravity at 27°C is given by:

  $$\mathrm{G_s} \text{ at } 27^{\mathrm{o}}\mathrm{C} = \mathrm{K} \text{ x Specific gravity at } T_x{}^{o}C$$

  Where:

  $$\mathrm{K} = \frac{\text{Density of water at temperature } \mathrm{T_x}{}^{\mathrm{o}}\mathrm{C}}{\text{Density of water at temperature } \mathrm{T_i}{}^{\mathrm{o}}\mathrm{C}} \tag{3.14}$$

3.4.15 Direct Shear Test

a) Objective

The objective of the test was to determine the angle of internal friction and cohesion for a fine, dry sand under direct shear boundary conditions.

b) Reference Literature

- BS 1377: Part 7: 1990
- ASTM D3080 / D3080M - 11: 2011

c) Significance

- The direct shear test was suited to the relatively rapid determination of consolidated drained strength properties because the drainage paths through the test specimen were short, thereby allowing excess pore pressure to be dissipated more rapidly than with other drained stress tests.
- The test was applicable on all undisturbed, remoulded or compacted soil materials, in assessing the strength in a field situation where complete consolidation had occurred under the existing normal stresses; and the results from several tests were used to express the relationship between consolidation stress and drained shear strength.

- The fixed location of the plane in the test could be an advantage in determining the shear resistance along recognisable weak planes within the soil material and for testing interfaces between dissimilar materials.

d) Apparatus

- Direct shear box apparatus
- Loading frame (motor attached), and Dial gauge
- Proving ring, Tamper, and Straight edge.
- Balance to weigh up to 200 mg, and spatula.
- Aluminium container

e) Procedure

i) The density bottle was cleaned by washing with water and dried by allowing it to drain.

ii) The inner dimension of the soil container was checked, and the parts of the soil container assembled together.

iii) The volume of the container was calculated, and its weight taken.

iv) The soil was placed in smooth layers (approximately 1cm thick), however whenever a dense sample was desired, the soil was tamped.

v) The soil container was weighed, and the difference of these two weights yielded the weight of the soil, from which the soil’s density was calculated.

vi) The surface of the soil was made plane.

vii) The upper grating was placed on stone and the loading block placed on top of the soil.

viii) The thickness of soil specimen was measured.

ix) The desired normal load was applied, and the shear pin removed.

x) The dial gauge was attached to measure the change of volume.

xi) The initial reading of the dial gauge and calibration values were recorded.

xii) Before proceeding to test, all adjustments were checked to see that there was no connection between the two parts except sand/soil.

xiii) The motor was started, and the reading of the shear force taken and recorded. Also, the volume change readings were taken until failure.

xiv) A 5 kg normal stress was added, and the experiment continued until failure.

xv) All readings were carefully recorded, and the dial gauges set to zero before starting the experiment.

xvi) The shear stress ($\tau = P_h/A$), normal stress ($\sigma_n = P_v/A$), horizontal displacement (δ_h) and vertical displacement (δ_v) for each observed value were computed.

xvii) The plotting of shearing stress (τ) against horizontal displacement (δ_h) and obtaining the maximum value of the shearing stress (τ_{max}) was made.

xviii) A graph of normal displacement versus shear displacement was plotted.

xix) A graph of the shearing stress (τ) against normal stress (σ_n) was plotted, whereby the y-intercept of the straight line gave the cohesion (c), and the angle of internal friction (∅) of the soil was determined from the slope by:

$$\emptyset = \tan^{-1}\left(\frac{\tau}{\sigma'_n}\right) \qquad (3.15)$$

f) Expected Results

- The parameters of the angle of internal friction (∅) in degrees, and the cohesion or intercept (c) in kg/cm^2 of the given soil sample tested in the laboratory.

3.4.16 One-dimensional Consolidation Test

a) Objective

The objective of the test was to determine the magnitude and rate of consolidation of soil when it was restrained laterally and drained axially while subjected to incrementally applied controlled-stress loads; which data was used in predicting the rate and amount of settlement of structures founded on clay.

b) Reference Literature

- BS 1377: Part 6: 1990
- ASTM D2435/D2435M - 11: 2011 or AASHTO T 216-07: 2012

c) Significance

- The data from the consolidation test was used to estimate the magnitude and rate of both differential and total settlements of a structure or earth fill. Hence, data estimates from the consolidation tests were of key importance in the design of engineered structures and the evaluation of their performance.

d) Apparatus

- Load device for applying vertical loads or total stresses to the specimen
- Consolidometer, and Specimen trimming device
- Balance sensitive to 0.01g, and Stopwatch
- Moisture can, and Drying oven

e) Procedure for preparation of samples and test specimen

i) The sample’s covering was removed, and its orientation maintained before placing the sample on a wax paper disc and glass plate.

ii) The diameter was rough-cut using a wire-saw to within 3.18 mm of the final diameter, and the sample's moisture contents got.

iii) The sample was assembled in the trimmer and trimmed using a cutting shoe and spatula for use in the second moisture content determination.

iv) Once the sample was completely fitted into the specimen ring, its top and bottom were trimmed with a wire-saw, and any final cuts to the top surface made with a sharp straight edge.

v) The 3rd and 4th moisture contents were obtained from these samples.

vi) Using the recess tool, space was created at the top of the ring, and all excess bottom soils trimmed and levelled with a sharp straight edge.

vii) The mass of the specimen and ring (M_{s+r}) was determined, and the recess from the top of ring to the soil surface (ΔH_i) measured.

f) Procedure for apparatus calibration

i) The cell was assembled, namely the stones, filter paper and top cap, before being aligned in the loading frame.

ii) A 453.5g seating load was placed on the cell and zero reading on the displacement transducer obtained.

iii) The same loads were applied to the apparatus as would be used in testing the specimen.

iv) At each load increment, the displacement readings at 15 sec, 30 sec, 1 min, 2 min and 5 min were recorded, with the change in dial reading giving the machine deflection curve.

g) Procedure for apparatus preparation

i) The Oedometer machine's assembly of stones, filter paper and top cap, and the initial height between top of cap and ring (z_3) were made.

ii) Then the Oedometer machine was disassembled; and the specimen ring and cutting shoe greased.

iii) The mass of the empty specimen ring (M_r), ring's height (M_r), ring's diameter (D_r), and the thickness of one piece of filter paper (H_{fp}) were determined; and then the two pieces of filter paper were cut.

iv) The stones were boiled for 10 minutes to clean and remove air.

v) Two (2) wax paper disks were cut to the diameter of the specimen.

h) Procedure for apparatus assembly

i) The base was water-filled, followed by insertion of the bottom stone; covered by filter paper and excess water removed with paper towel.

ii) The specimen and ring were placed on the stone, the rim covered with a gasket and tightened with locking ring.

iii) The specimen was covered with filter paper and a top stone, and the stone allowed to drain before placing on the soil.

iv) Thereafter, the top cap was placed on the stone, and the height (z_3) measured with the specimen.

v) The assembly was located in the loading frame with dial gauge and balance arms, and the 453.5g seating load applied.

i) Test Procedure

i) The specimen was consolidated using a load increment ratio ($\Delta P/P$) between 0.5 and 1.0 for loading and -0.25 and -0.50 for unloading.

ii) The recommended schedule (S) being used was 0.125, 0.25, 0.5, 1.0, 2.0, 4.0, 8.0, 4.0, 1.0.

iii) The water bath was filled at 25% of the overburden stress of about 0.25 kilo-pound per square inch (ksi) or 1723.7 kPa or within 2 hours.

iv) For each increment, the displacement transducer reading versus time was recorded, remembering that the initial portion of the curve was very important to define the start of consolidation (ε_s).

v) During each increment, a plot of both root time and log time curves were made, with additional increments applied after the end of primary consolidation had been reached.

vi) One cycle of secondary compression was allowed to occur under maximum load and before the unload-reload cycle; and at the end, the specimen was unloaded to seating load and allowed time for swelling.

vii) The water was removed from the bath and so was the specimen from the apparatus. After, any extruded soil was removed and oven dried.

viii) The specimen surface was dried and mass of both soil and ring determined; thereafter, the soil extruded, and its moisture content got.

ix) The washings from the filter paper and the inside of the ring were collected and oven dried.

j) Expected Results

- The calculations and results from the following equations (3.16-3.25):

Initial Specimen Height = $H_r - \Delta H_i - H_{fp}$

$$\text{Moisture content} = \frac{(\text{Total mass} - \text{Dry mass})}{\text{Dry mass}} \quad (3.16)$$

$$\text{Void Ratio} = \frac{(\text{Total volume} - \text{Volume of solids})}{\text{Volume of solids}} \quad (3.17)$$

$$\text{Volume of solids} = \frac{\text{Mass of oven dried soil}}{\text{Specific Gravity}} \tag{3.18}$$

$$\text{Degree of saturation} = \frac{\text{Specific Gravity x Moisture content}}{\text{Void ratio}} \tag{3.19}$$

$$\sigma'_v = \frac{\text{Applied load} - \text{tare load} + \text{top cap and stone}}{\text{Area}} \tag{3.20}$$

Where:

σ'_v = Vertical effective stress when the pore pressure is zero

$$\varepsilon_v = \frac{\text{measured axial deformation} - \text{apparatus compression}}{\text{Initial specimen height}} \tag{3.21}$$

Where:

ε_v = Vertical strain

$$\text{Compressibility} = \mathrm{a_v} = \frac{-\text{ change in void ratio}}{\text{Change in vertical stress}} \tag{3.22}$$

$$\mathrm{C_v}(\text{Root time}) = \frac{0.848 \text{ x (drainage height)}^2}{\text{time for 90\% consolidation}} \tag{3.23}$$

$$\mathrm{C_v}(\text{Log time}) = \frac{0.197 \text{ x (drainage height)}^2}{\text{time for 50\% consolidation}} \tag{3.24}$$

Where: $\mathrm{C_v}$ = Coefficient of consolidation

And the drainage height is computed at 50% consolidation for both cases.

$$\text{Hydraulic conductivity} = \mathrm{k_v} = \left[\frac{\mathrm{C_v}\ x\ \mathrm{a_v}\ x \text{ unit weight of water}}{1 + \text{average void ratio}}\right] \tag{3.25}$$

$\mathrm{C_a} = \Delta$ in strain per log cycle of time after the primary is complete.

Where: $\mathrm{C_a}$ = rate of secondary compression

- Tables and graphs of time vs vertical dial reading, Void ratio-pressure, and coefficient of consolidation calculations.
- Corresponding plotted graphs of Dial readings vs time, void ratio vs log of pressure, and coefficient of consolidation vs logarithm of pressure.

3.4.17 Soil Resistivity Test

a) Objective

The objective of the Soil Resistivity Test or Geophysical surveys (Wenner Four-Electrode Method) was to determine the conductivity of the soil in ohm meter.

b) Reference Literature

- ASTM G57: 2001
- IEEE Std 81: 2012; and
- BS 7430: 1998

c) Significance

- Soil resistivity influences the plan of an earthing system and is the major factor that decides the resistance to earth of a grounding system. Thus, before designing and installing a new grounding system, the determined location was tested to find out the soil's resistivity.
- Soil resistivity was considered to be a preliminary indicator of the soil's corrosivity and aided in the identification of potential corrosion causing hazards in soils and water, since resistivity is a function of soil moisture and the concentrations of ionic soluble salts. (Roberge, 2000)

d) Apparatus

- Earth Resistivity Tester/Injection test unit
- Ground Grid Conductor Locator
- Flexible Insulated Wire on Easy-to-Spool Reels
- Earth test electrodes (stake), and 240V Portable Generator
- Measuring Wheel/Measuring tapes (100m), and Hammers

e) Procedure

i) The Standard split-barrel Sampler (Split-spoon) was attached to the bottom of the drilling rod, while the top of the drilling rod attached by anvil was used to transfer the hammer load to the drilling rod. The anvil was connected to a guide rod passing through the drop hammer.

ii) The distance required between the pins namely the electrode spacing (a) was in the order 0.3 m, 1.0 m, 2.0 m, 3.0 m, 4.0 m and 5.0 m was measured using PASI 16 GL-N - P100-XN_LP equipment.

iii) The pins were placed in the ground.

iv) The test-leads were connected to the designated pins and earth tester terminals.

v) The earth tester was operated according to the manufacturer's instructions.

vi) The measured resistance was recorded.

f) Expected Results

- The measured resistance value (Ω) and the calculated value of apparent soil resistivity (ρ) using the Wenner (1915) equation:

$$\rho = 2\pi\frac{\Delta V}{I} = 2\pi aR \quad (3.1)$$

Where:

$\rho =$ is the apparent resistivity of the soil in Ω

$R =$ is the measured resistance of the soil in Ω

$a =$ is the electrode spacing in metres

$\Delta V =$ voltagemeasured (volts), and

$I =$ injected current (Amps)

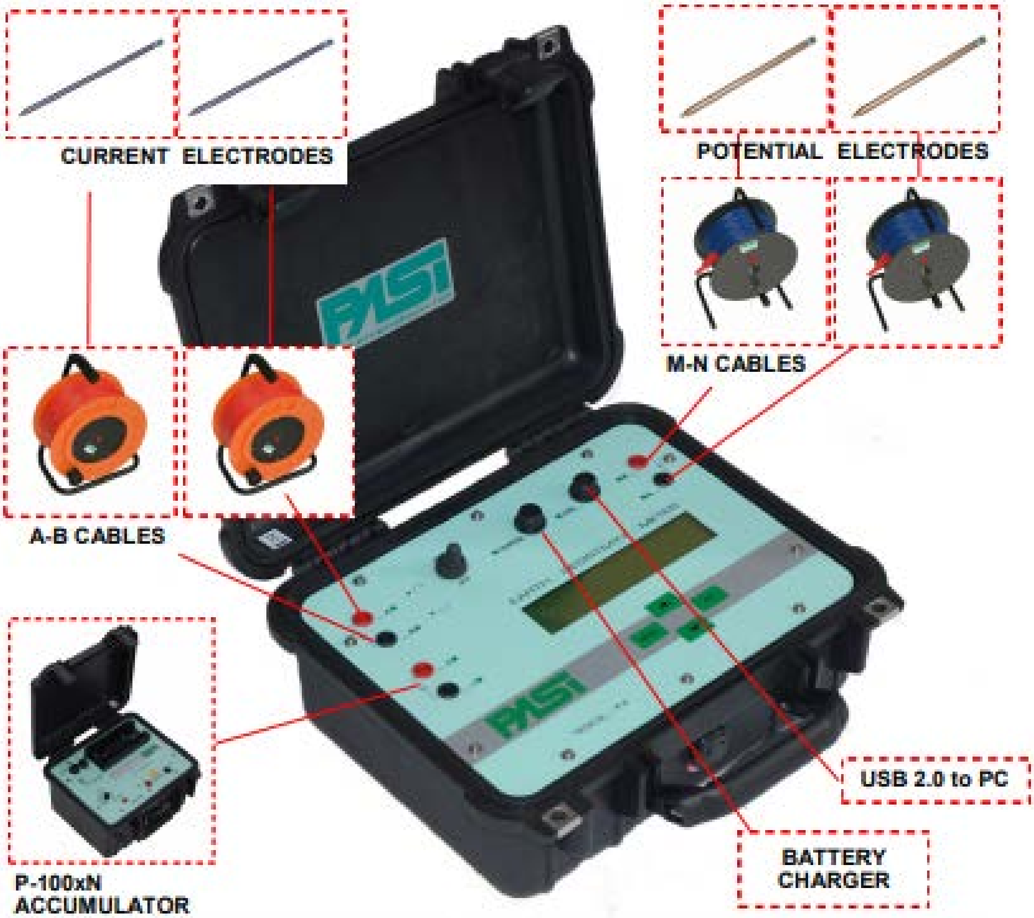


Figure 3.4: Standard Earth Resistivity Meter

Figure 3.5: Soil resistivity measurement in the field

3.5 Geotechnical Foundation Design

3.5.1 Evaluation of bearing capacity based on corrected SPT N-values

3.5.1.1 Terzaghi's approach (1967)

The allowable bearing capacities, q_{all} were computed using the corrected SPT N'_{55} values from equations (2.14) to (2.16):

$$\Rightarrow N'_{55} = \left(\frac{p''_o}{\gamma' \text{ x depth}}\right)^{\frac{1}{2}} \text{x N x } \frac{E_r}{E_{rb}} \text{ x } \eta_2 \text{ x } \eta_3 \text{ x } \eta_4 \tag{2.14}$$

$$\Rightarrow q_{ult} = 5.14 \text{ x} \frac{q_u}{2} = \left[5.14 \text{ x} \frac{13.1 \text{ x } N'_{55}}{2}\right] \tag{2.15}$$

$$\Rightarrow q_{all} = \frac{q_{ult}}{FS} = (5.14 \text{ x } c_u)/FS = \left\{5.14 \text{ x} \left[\frac{(13.1 \text{ x } N'_{55})}{2}\right]\right\}/FS \tag{2.16}$$

3.5.1.2 Bowles's approach (1982)

For Bowles' method, equations (2.17) to (2.23) were used as follows:

$$q_a = \left\{\frac{N}{F_2}\left[\frac{(B+F_3)}{B}\right]^2 x \text{ } K_d\right\} \text{for B} > F_4 \tag{2.17}$$

$$q_a = \left\{\frac{N}{F_1} x \text{ } K_d\right\} \text{for B} \leq F_4 \tag{2.18}$$

$$N'_{55} = C_N \text{ x N x } \eta_1 \text{ x } \eta_2 \text{ x } \eta_3 \text{ x } \eta_4 \tag{2.19}$$

$$q_{all} = 0.73 \text{ x } N'' \text{ x } R_{D_1} \text{ x } S_a \text{ [kN/m}^2 \text{ for B} \leq 1.2\text{m]} \tag{2.20}$$

$$q_{all} = 0.48 \text{ x } N'' \text{ x } R_{D_2} \text{ x } \left(\frac{B + 0.3}{B}\right)^2 \text{ x } S_a \text{ [for B} > 1.2\text{m]} \tag{2.21}$$

$$R_{D_1} = 1 + 0.2\left(\frac{D_f}{B}\right) \leq 1.2 \text{ for } \emptyset = 0 \tag{2.22}$$

$$R_{D_2} = 1 + 0.1\left(\frac{D_f}{B}\right) \leq 1.2 \text{ for } \emptyset = 0 \tag{2.23}$$

3.5.2 Pile foundation capacity

3.5.2.1 Pile skin resistance capacity

For clay soils, a general method for pile shaft skin resistance established by Poulos (1980) was used as follows:

$$f_s = \alpha S_u \tag{3.26}$$

Where:

$\mathrm{f_s} = \text{is shaft skin resistance, implying } \mathrm{f_s} = 100 \text{ kPa maximum}$

$\alpha = \text{adhesion factor}$

$\Longrightarrow\ \alpha = 0.45 \text{ (for non} - \text{fissured clays)}$

$\Longrightarrow \alpha = 0.3 \text{ for fissured clay in bored piles}$

$S_u = \text{average undrained shear strength}$

$$S_u = \frac{q_u}{2} \tag{3.27}$$

Where:

$q_u = \text{unconfined compressive strength}$

$$\Longrightarrow q_u = 12 \text{ x corrected SPT value } (N_{55}) = 12\ x\ N_{55} \tag{3.28}$$

For cohesionless soils, the equation developed by Bruland (1973) was used for skin resistance calculations as shown in equation (3.29):

$$f_s = K\bar{q}\tan\delta \tag{3.29}$$

Where:

K = lateral earth pressure coefficient

K = $(1 - \sin\emptyset')$ from Jáky's (1944, 1948) semi-empirical expression for the coefficient of earth pressure at rest

$\emptyset' = \text{the effective angle of internal friction, derived from SPT results}$

$\bar{\mathrm{q}} = \text{effective overburden pressures}$

$\delta = \text{friction angle, which is equal to effective angle of internal friction}$

The correlation between normalised blow-count $(N_1)_{60}$ and $\emptyset'$ was established by equation (3.30):

$$(\mathrm{N_1})_{60} = \frac{\mathrm{N_{55}\ x\ 55}}{60\ \mathrm{x}\ \eta_2} \tag{3.30}$$

Where:

η_2 = the rod length correction

N_{55} = corrected SPT value

3.5.2.2 Pile end bearing resistance

Poulo's (1980) relation was used for end bearing resistances (for clay), whereas for sands/gravels, Meyerhof's equations (1976) were used, as shown in Table 3.2:

Table 3.2: End bearing of piles (Bowles, 1997)

Soil Type	Relationship	Values	
		Bored Piles	**Driven Piles**
Clay	$q_b = N_c S_u \omega$	$N_c = 9$	$N_c = 9$
		$\omega = 1.0$ (Not Fissured)	$\omega = 1.0$
		$\omega = 0.75$ (Fissured)	-
Sand	$q_b = N_q \bar{q}$	$N_q = 20$ (Loose)	$N_q = 20$ (Loose)
		$N_q = 30$ (Medium Dense)	$N_q = 30$ (Medium Dense)
	$q_b = 10\ MPa$ (Maximum)	$N_q = 60$ (Dense)	$N_q = 60$ (Dense)
		$N_q = 100$ (Very Dense)	$N_q = 100$ (Very Dense)

Based on the above equations, the values for pile skin-resistance and end-bearing resistance recommended for design for bored piles were calculated and recorded in Appendix I (Foundation design calculations for AP 104/5), where the values of effective overburden pressure took account of the backfill.

For any given depth, values of $\alpha, N_{55}, N_c, \omega,$

Where: α = adhesion factor

$\Longrightarrow$ $\alpha = 0.45$ (non-fissured clays); and $\alpha = 0.3$ (fissured clay in bored piles)

N_{55} = corrected SPT value; and N_c = Terzaghi's bearing capacity factors

For cases where clay was the predominate particle, the following values were used:

$$f_s = \alpha S_u = \alpha\left(\frac{q_u}{2}\right) = \alpha\left(\frac{12 \text{ x } N_{55}}{2}\right) \tag{3.31}$$

$$q_b = N_c S_u \omega = N_c\left(\frac{q_u}{2}\right)\omega = \left[N_c\left(\frac{12 \text{ x } N_{55}}{2}\right)\omega\right] \tag{3.32}$$

Where:

f_s = Shaft skin resistance; and S_u = average undrained shear strength

α = Adhesion factor; and q_u = unconfined compressive strength

q_b = End bearing resistance; and N_c = Terzaghi's bearing capacity factors

For a depth with predominate soil particles as sand, the value of effective angle of internal friction was obtained based on the relationship among $(N_1)_{60}$, density index (I_D) and the effective angle of internal friction (ϕ') as follows:

$$f_s = K\bar{q}\tan\delta = [(1 - \sin\emptyset')\ x\ \bar{q}\ x\ \tan\emptyset']$$

$$\Longrightarrow f_s = [1 - (\sin\emptyset'\ x\ \bar{q}\ x\ \tan\emptyset')]\ (in\ radians) \tag{3.33}$$

$$q_b = N_q\bar{q} \tag{3.34}$$

Where:

f_s = Shaft skin resistance

K = lateral earth pressure coefficient; $\bar{q}$ = effective overburden pressures

δ = friction angle, which is equal to effective angle of internal friction

$\emptyset'$ = the effective angle of internal friction, derived from SPT results

q_b = End bearing resistance; and N_q = Terzaghi's bearing capacity factors

3.6 Structural Foundation Design

3.6.1 Designing codes used

Various design codes were referenced during the prescriptive structural and geotechnical designs and analyses, as briefly described in the following sections.

3.6.2 Steel Lattice Tower Designs

Standards of the International Electro-technical Commission (IEC) and the International Standardisation Organization (ISO) or as referenced in the KIP's technical specifications (IEEE-691, 2001; IS 1200-1, 1974; MoE & MD, 2013) were used such that all steel lattice tower designs and detailing were to the requirements of the American Society of Civil Engineers' Standard (ASCE 10-1997) for the Design of Latticed Steel Transmission Structures.

3.6.3 Structural and high strength steel materials

All structural and high-strength steel materials hot-rolled were designed to conform to the steel qualities S 235 JO and S 355 JO respectively according to BS EN 10025: 1990 + A1: 1993 or BS EN 10210 as per BS EN 50341: 2001.

All bolts and nuts with hexagonal heads, flat and spring washers for securing tower members and parts complied to BS 4190, and/or BS EN 20898.

3.6.4 Structural Steel Galvanisation

All steelworks (hot-dip galvanised) were in accordance with BS 729, ASTM A-123 and ASTM A-153 to provide a smooth. clean and uniform zinc coating of minimum $10g/m^2$, and a 86μm thickness for bars, plates, bolts, except threaded work where a uniform minimum zinc coating of 500 g/m^2 was used.

3.6.5 Reinforced concrete design

The design of reinforced concrete structures was in accordance with BS 8110 as per the requirements of the Technical Specifications (IEEE-691, 2001; IS 1200-1, 1974; MoE & MD, 2013) for transmission works to assess the internal stresses in concrete and in steel reinforcements.

The approved steel bars from the KIP stores were cut and bent as per BS EN 4466, equivalent to ISO 4066 and BS 8666; and complied with the following standards in other respects:

- BS 4449 for hot rolled steel bars.
- BS 4461 for cold worked steel bars.
- BS 4482 for hard drawn mild steel wire.

3.7 Foundation full-scale model construction

3.7.1 Site clearance and setting out

The first task was to check that the permitted area of work arranged by the EPC contractor was verified, cleared of all vegetation and obstructions so that it was suitable for safe and proper placement of construction material and for performing the foundation activity. During setting out, the centre and adjacent line pegs were used as reference for setting out works.

3.7.2 Pit Marking

After completion of the soil classification, geotechnical and structural design works, the limits of excavation were marked by placing excavation pegs with reference to the appropriate foundation drawings.

3.7.3 Foundation excavation

Excavations of the shallow ‘pad and chimney’ foundations were executed both manually and by use of an excavator. Care was taken to keep the excavated soil at a minimum safe distance of 1.5m from the pit edges as per the technical specifications to avoid collapse of soil into the pit or exertion of extra overburden pressure on the pit. Where there was a possibility of collapse of the excavation at a depth of 1.5m or more, shoring was used as per the technical requirements of IEEE-691 (2001) and IS 1200-1 (1974) for transmission lines in Uganda (Kim and Cho, 1995; Jang *et al*., 2007; Bayliss and Hardy, 2011; MoE & MD, 2013; Sriram and Prasad, 2017). Upon excavation completion, final levels and dimensions were checked and recorded using a dumpy level and measuring tapes, respectively.

3.7.4 Lean concreting application

A 50 mm thick lean concrete was laid after removing all loose material from the pit, to provide a clean and level working surface, and the lean concrete thickness was maintained by fixing steel pegs to the required level.

In case of extra excavation in depth and length, the extra dimensions were filled with lean concrete and not the use of compacted excavated fill soil.

3.7.5 Stub setting

The stubs were set using props and care was taken to firmly anchor the prop plates on to the ground. The ‘turn-buckles’ in the prop arrangement were checked and oiled to enable free movement.

3.7.6 Reinforcement assembly

Reinforcement steel bars of sizes in the drawings were cleaned of foreign material, assembled while maintaining the approved rebar spacing and secured using binding wires. As per design, a concrete cover of 100 mm and 50 mm was maintained to reinforcement bars at the bottom and to all other surfaces, respectively.

3.7.7 Formwork assembly

Steel formworks were used for the works, with all connections at the corners checked to prevent mortar from flowing out of the bases. The formwork thicknesses were checked to prevent issues of sagging, cleanliness, and oiling before use.

3.7.8 Concrete casting

Concrete mixing was done insitu by volume batching, and the top surface of the lean concrete cleaned of all debris, and moist with cement slurry in preparation for concrete casting. All fine and coarse aggregates were supplied from approved sources as per the technical specification requirements. The approved project design mix for cube strengths of not less than 25 N/mm^2 for 28 days' strength was followed. The concrete batch of aggregates, cement and water were mixed homogenously in a mixer machine for a minimum of 5 minutes, with every batch being tested for slump checks. Concrete cubes were randomly taken for testing for 7-day and 28-day strengths. To prevent segregation of the constituent concrete material, proper care was taken to transfer the concrete from the mixer to the pit with adherence to the following:

- Maximum free fall height of concrete was limited to less than 2m depth with the use of chutes for gentle pouring of concrete.

- During pouring, the concrete was vibrated using concrete vibrators to ensure proper compaction; the vibrators were not used as a means of repositioning the placed concrete.

Concreting of ‘pad and chimney’ of one leg was done in the same day, commencing with the bottom pad, followed by the chimney column sections and the capping.

3.7.9 Additional quality assurance and quality controls

(a) Formwork removal

The formworks were removed after a minimum of 24 hours after concreting as per the requirements of the technical specifications for transmission works.

After carefully removing the formworks, the concrete surfaces were checked and repaired for any imperfections such as honeycombing.

(b) Concrete curing

Following the removal of formworks, all the exposed concrete surfaces were protected by application of moist jute covers with continued application of curing water for a minimum of 7-days until the commencement of backfill.

(c) Protection of foundation concrete and stubs

For locations with higher levels of sulphate or chloride, all stubs and concrete surfaces were coated with two coats of an approved bituminous paint.

(d) Backfilling

Suitable backfill material to be compacted were ensured to be well-graded and containing no stones greater than 100 mm in any of its dimensions; and the approved backfill material was laid in layers of 200 mm, with each layer compacted to 91% MDD using mechanical means.

(e) Cube test

During concrete casting, six (6) concrete cubes of 150 x 150 x 150 mm sizes were prepared on site in the mould in 50mm layers, with each layer being thoroughly compacted with a 380 mm long, 25 mm^2 ramming face, and 2.8 kg smooth steel rod. After 24 hours, cubes were transferred to the store for curing and subsequent testing as per BS EN 12390-1 (2012) and BS EN 12390-2 (2009) to check the concrete cube strengths in conformity with the required 1.4 kg/mm^2 (13.72 N/mm^2) for 7 days and 2.5 kg/mm^2 (25 N/mm^2) for 28 days, respectively. Results of concrete cube tests are in Appendix C.1.

(f) Site clean-up

After completion of all foundation activities, the sites were cleaned of all surplus excavated material and construction materials; and the work sites were reinstated wherever possible, to the original ground contours.

3.8 Static Load Test methodology

3.8.1 Test Procedure

The constructed full-scale foundation models were tested to assess their insitu load carrying capacities and the foundation load responses to the design load as per IEC 61773:1996 for overhead lines testing of foundation structures.

3.8.2 Reference Literature

- ASTM D3689: 2013 (Static Axial Tensile (or uplift) Load Test)
- ASTM D1143: 2013 (Static Axial Compressive Load Test)
- ASTM D3966: 2013 (Lateral Load Test)
- IEC 61773:1996 or COMESA/FDHS 293: 2007

3.8.3 Foundation loads applied

The support system consisted of a 7m long reaction beam supported either side on sets of trapezoidal precast concrete slabs for withstanding any expected deformations. It was arranged to achieve an 8° inclination angle. The Appendix D details the graphs of load vs time, load vs displacement, and displacement vs time.

3.8.4 Test Apparatus

Following apparatus were used among others:

- Test loading beam, and Reference beams
- Hydraulic jack, and Micrometre
- Hydraulic pressure gauge

3.8.5 Arrangement of Apparatus

a) Placement of Testing Beam

As per the specifications, the test loading beam was checked to be strong enough to take the test load of at least 1.5 times the design load applied. The reaction system was placed on a hard base to provide the required support, and at a suitable distance to prevent the influence of foundation uplift. The stub was connected to the loading beam by a system of bolts. The hydraulic pressure gauge that was fitted to the hydraulic jack measured the load applied. The entire test apparatus system was arranged and designed in such a way that it could not alter the prescriptively designed behaviour of the foundation while the load was being applied. The clear distance between the reaction supports was more than the minimum allowable distance (L) as given by:

$$L = B + 0.7D \text{ (For Pad and Chimney Foundations)} \tag{3.35}$$

Where:

L = Nearest point between points of reaction supports

B = Width of foundation; and D = Depth of foundation

The clear distance between the reaction supports and the test pile-foundation was more than the minimum distance (L) given by:

$L = 3e$ or 2, whichever was greater (For Piled Foundations) (3.36)

Where: e = diameter of the test piled foundations

b) Placement of reference beam

The reference beam was assembled stiff enough to resist the instrumentation without excessive deflection. The dial gauge was connected to the reference beam, and the machined plates fixed by bolting to the foundations to provide a smooth surface to measure the displacements. The depths of the supports to the reference beams were 1m to 3m depending on soil type, and whereas in rock, it was reduced so that the vertical and lateral movements were restrained.

The reference beam support was maintained to be not less than (C):

$$C = 0.35D + 0.5 \quad (3.37)$$

Where:

C = support of reference beam distance to edge of foundation

D = depth of foundation

Support of reference beam was maintained not less than either of:

$C = 1.0\,e + 0.5$ or 1.5, whichever was greater (3.38)

$C = 2.0 + 0.5\,e$ (in metres), for laterally loaded piles (3.39)

Where:

C = support of reference beam distance to edge of foundation

e = diameter of test pile foundation

3.8.6 Assembly of foundation test setup

The figures below show the schematic layout of the test foundations.

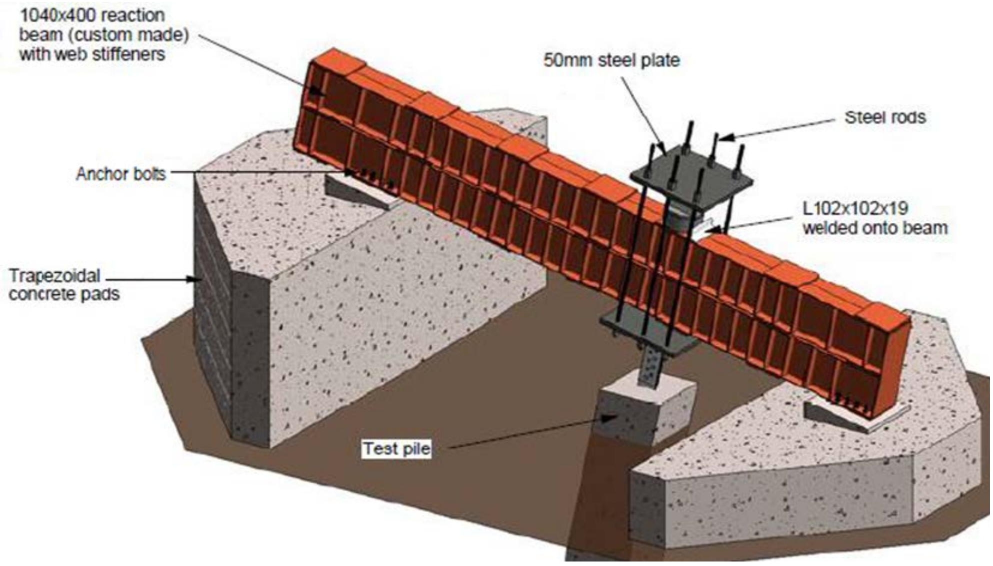


Figure 3.6a: Static Axial Uplift/Tensile Load Test (MoE & MD, 2013)

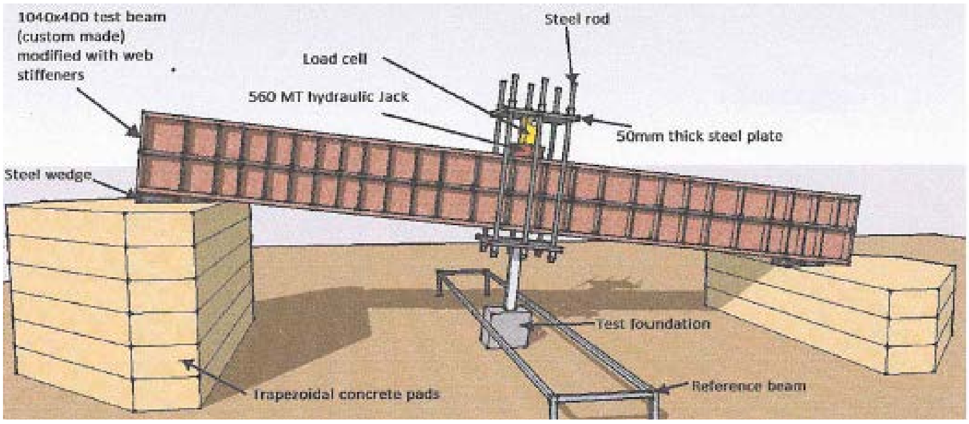


Figure 3.6b: Static Axial Uplift/Tensile Load Test (MoE & MD, 2013)

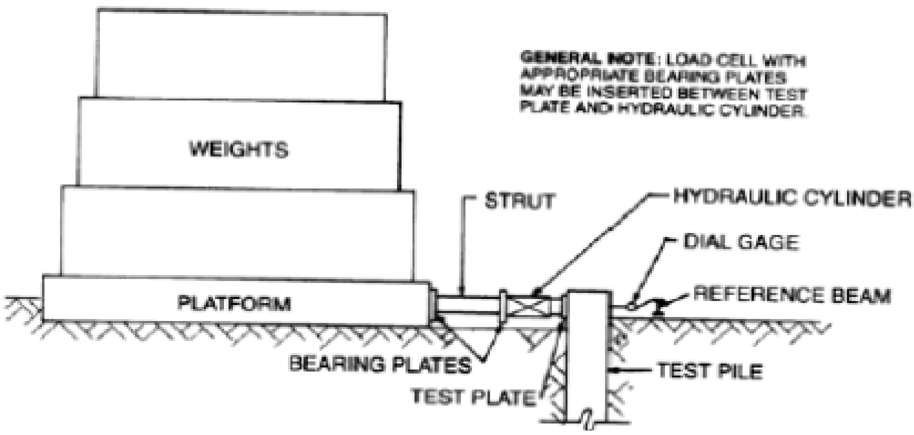


Figure 3.7: Lateral Load Setup (FHWA-SA-91-042, 1992)

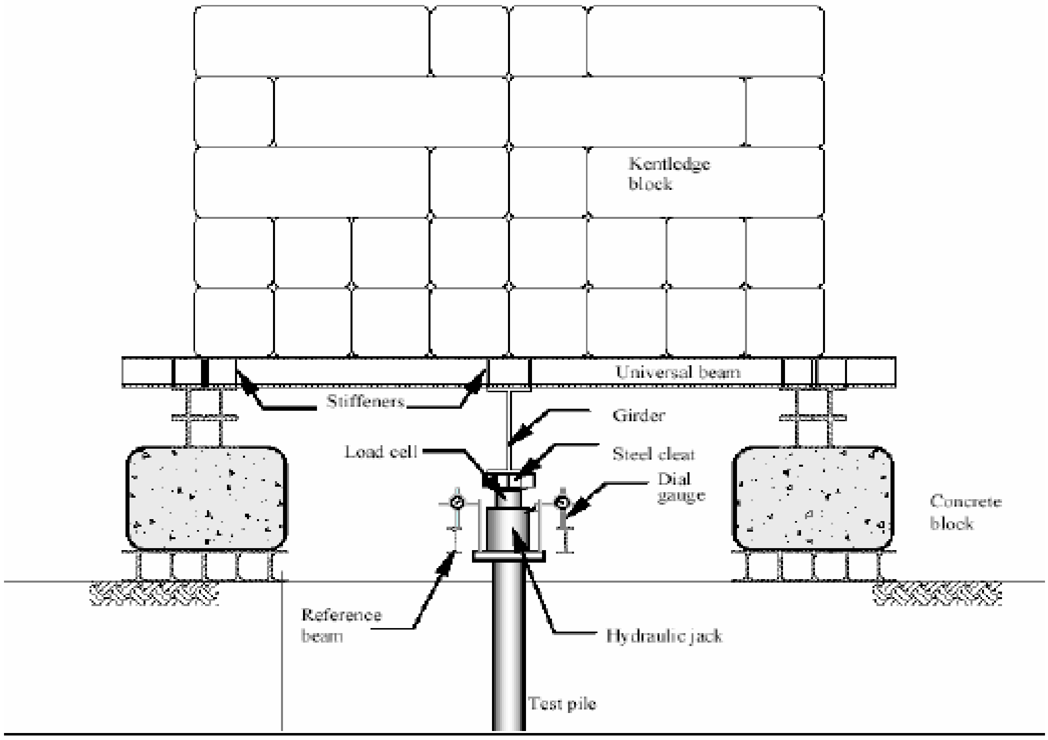


Figure 3.8: Compressive Load Test (FHWA-SA-91-042, 1992)

The elevations of the upper 50 mm thick metal plate above the reaction beam and the lower 50 mm thick metal plates were established to enable the proper configuration of the loading hydraulic jack, the load cell/column and test foundation to be set up taking into consideration the limit uplift distances and reaction beam deflections. The loading hydraulic jack was a HI-FORCE jack with a capacity of 500 ton. An electric pressure pump connected to the loading jack to supply the load to the jack was utilised. A Macklow Smith load cell was placed between the jack and the upper 50mm thick metal plate to read off the load being applied. Three displacement dial gauges were fixed on a reference frame with support rods and rested on glass plates placed over the foundation stub. The reference frame was designed, fabricated, and placed as per the guidelines of IEC 61773 (1996). The plans and elevations of the test foundations, test apparatus, reaction systems, and fixed reference points are shown in Appendix D.

3.8.7 Load application

The load to the hydraulic jack was applied by a pressure pump fitted with a pressure gauge. A load cell was placed between the jack and the upper 50 mm thick metal plate. The load cell was mainly used to measure and monitor the amount and stability of force applied to the test beam. The upper 50 mm thick metal plate was connected to the lower 50 mm thick metal plate below the reaction beam, which was in turn connected to the double angle metal connection of the foundation stub applying the loading force. In the static axial uplift load test arrangement, the reaction beam was resisted from movement by the trapezoidal concrete support slabs or support piles. This arrangement, therefore, resulted in the load being applied by the jack to lift the test foundation, and the resulting displacement in the foundation measured by the displacement gauges.

3.8.8 Test Procedure

After the test cubes of the insitu foundation had passed the required design compressive strength after 28 days, the test pile was then tested.

During the static uplift/tensile, compressive, and lateral load tests, the loads were applied axially to the test foundation bases and columns/‘chimneys’.

a) Loading Procedure

Initially, a load of 10% of design load was applied to check the stability of the test equipment. Load was applied in steps, in percentage (%) of the target load of 25, 50, 70, 80, 90, 100; and the loads were maintained for 10 minutes for each loading step. However, the designed load was maintained for 30 minutes to check that no significant movements had occurred in the foundation. Further load increments of 10% were made beyond the design loads until failure point or up to 130% (whichever occurred first), with each loading step maintained for 3 minutes. However, for cases of cohesive soils, the loads were maintained for 30 minutes each starting from the loading steps of 70% and above.

The application of the load was in stages of 25% and 10% of working load in the loading sequence. Loading was applied to 130% of the working load in the sequence of 25%, 50%, 70%, 80%, 90%, 100%, 110%, 120%, and 130%. Two units of measure were used to establish the amount of load being applied, namely pressure pump gauge and the load cell.

b) Measurement of Displacements

A reference frame was placed over the test foundation stub in addition to three (3) dial gauges equidistant from each other. The gauges were fixed to the reference frame and the displacement-measuring rod placed barely touching the glass plates on top of the foundation stub. The measurement of the foundation's displacements was carried out by reading the displacement on the dial gauges. Base readings at the commencement of the test were first taken after the application of pressure equivalent to less than 10% of the working load and just enough for making contact between the load cell and the test beam. Thus, subsequent readings were deducted from these base readings to establish the foundation's displacements. The displacement measurements were taken at 10-minute intervals for loads starting at 25% up to 90%, and at 30-minute intervals for the load at 100%, and at 3-minute intervals for loads at 110%, 120% and 130% of the working loads respectively. The loading cycle, records of tests such as field measurements and plots of the load vs time, load vs displacement, and displacement vs time, are all shown in Appendix D.

c) Primary measurement system

Mechanical dial gauges with a recommended resolution of 0.1 mm or less and a recommended range of travel of 50-150 mm, preferably 150 mm, were used for design and proof tests. The dial gauges were clamped to the reference beam in such a manner that the gauge expanded as the load was applied, in order to prevent damage to the instrumentation in the event of a sudden failure of the foundation or equipment. During the tests, a minimum of two gauges were mounted equidistant from the vertical foundation axis and from each other.

d) Secondary measurement system

As a control-check on the primary measurement system, a secondary measurement system was used in the test. A theodolite and graduated scale were used to measure the displacements in addition to the main dial gauges.

3.8.9 Test Evaluation

The test results were evaluated in relation to the “as-built “conditions; since prior to the tests, the load capacities of the foundations were prescriptively calculated using the parameters obtained from the geotechnical investigations.

a) Acceptance criteria

The acceptance criteria from the static load tests were based on the load and displacement results from design test achieving the requirements of IEC 61773 (1996) for the design values to be deemed as satisfactory. However, in cases of premature failure, the cause of failure would be reviewed and suitable modifications to design and procedure of test made.

b) The failure criteria for Loading tests

The test foundation was deemed to have failed under the static load test methods if the following occurred as per IEC 61773 (1996):

- Need for continuous jacking to maintain the load.
- When the uplift of foundation exceeded 25mm or the calculated values.
- When compression settlement of the foundation exceeded 25 mm.
- When lateral displacement of the foundation exceeded 50 mm.

c) Insitu foundation capacity determination

From the analysis of the hyperbolic graph, the derivative of the equation of the line of 'best-fit', was considered to be the slope (C_1) as shown below:

$$y = m\,x + c \qquad \text{(Equation of the line of best fit)} \tag{3.40}$$

$$Slope\ C_1 = \frac{dy}{dx} \tag{3.41}$$

According to IEC 61773 (1996) standard, the actual insitu load capacity (R_c) of the foundation was determined from the hyperbolic model graphs using the empirical equation of the Chin-Kondner Extrapolation (1971) shown below:

$$\text{Load Capacity} = \mathrm{R_c} = \frac{1}{\mathrm{C_1}}\ \text{(in kN)} \tag{3.42}$$

Table 3.3: Inclined foundation load test details

S/No	Inclined Foundation Load Test Details				
1	**Foundation number**	**KL 30 (B103+5)**	**AP 108/15**	**AP 108/20**	**AP 104/5**
2	**Foundation coordinate**	N426605.777, E246178.091	N194777.676, E391575.254	N192810.148, E391873.474	N212510.472, E400765.294
3	**Foundation tower type**	DB-Waterlogged	ST-Poor soil	DA-Good soil	DA-Pile
4	**Transmission Line**	132 kV	400 kV	400 kV	400 kV
5	**Foundation base level**	- 4.50m	- 2.75m	-3.50m	-12.775m
6	**Foundation base size**	4.5m x 4.5m	4.39m x 4.39m	2.64m x 2.64m	Ø 900 mm
7	**Stub top**	+0.5m	+0.5m	+0.5m	+0.5m
8	**Pile head size**	0.55m x 5m (inclined)	0.60m x 3.05m (inclined)	0.60m x 3.8m (inclined)	1.2m x 1.2m x 2.225m
9	**Stub angle**	6.84 degrees	8 degrees	8 degrees	90 degrees (Vertical)
10	**Working Load (WL)**	962.26 kN	727.20 kN	594.45 kN	555.69 kN (Tension), 828.52 kN (Comp) & 180.59 kN (Lateral)
11	**Maximum Test Load (130% of WL)**	1250.94 kN (127.56 ton)	945.36 kN (96.4 ton)	772.785 kN (78.8 ton)	722.397 kN (Tension), 1077.08 kN (Comp) & 234.77 kN (Lateral)
12	**Test Load Type**	Tensile	Tensile	Tensile	Tensile, Compressive & Lateral
13	**Loading system**	Reaction Beam	Reaction Beam	Reaction Beam	Reaction Beam
14	**Testing standard**	IEC 61773: 1996	IEC 61773: 1996	IEC 61773: 1996	IEC 61773: 1996

3.9 Schematic diagram for methodological approach

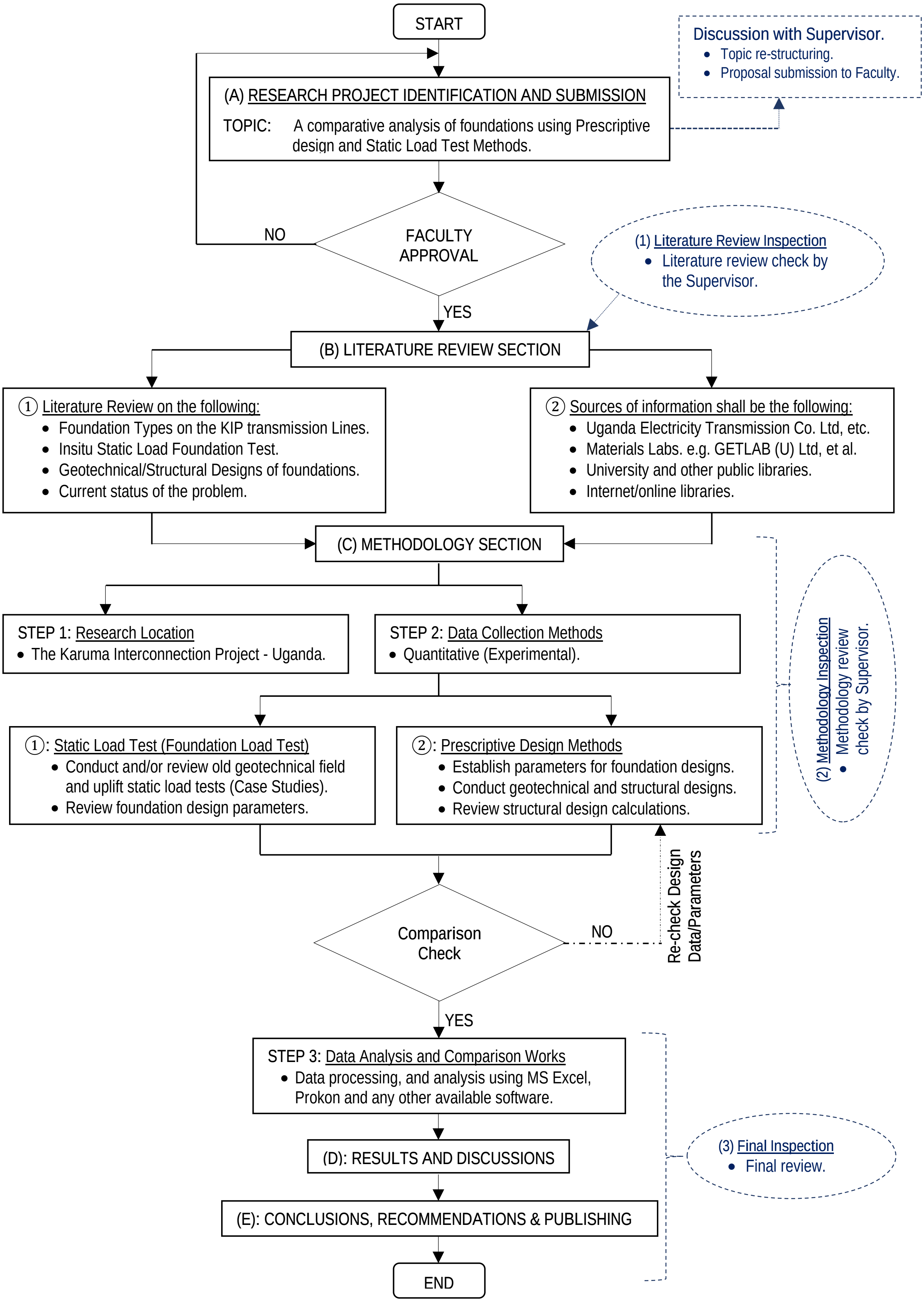


Figure 3.9: Schematic diagram for methodological approach

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Introduction

In this chapter, the results and analyses of field data from four (4) overhead transmission line test foundations are presented. Since the test foundation is a composite structure comprising the soil upon which the footing is anchored and the reinforced-concrete structure; therefore, the foundation's constitutive behaviour is governed by understanding some if not all of the soil's properties and foundation's load-bearing characteristics under full-scale static load test.

4.2 Insitu Geotechnical Tests

4.2.1 Test Trial Pits and Borehole Pits

Test trial pits and/or borehole pits were done to determine the location of ground water tables and soil strata descriptions as summarised in Tables 4.1 and 4.2 respectively, with more details found in the Appendices B.1 and B.2.

Ground water tables were encountered above the base of the footing in the swampy locations of KL 30 and AP 104/5 at levels of 0.3m and 1.14m respectively; and ground water tables were below the base of the footings in locations AP 108/15 and AP 108/20 at levels of below 10m and 4m respectively below existing ground level (Das and Sobhan, 2018).

The results from above water table were used in computations of the soil's effective unit weights (γ'), magnitudes of unit surcharge (q) and corrections for ground water table effects on bearing capacities as per equations 2.33 to 2.37, and subsequent bearing capacities as discussed in sections 4.2.2, 4.2.3, 4.3.2 and 4.3.3.

However, insitu soil profile descriptions in Appendices B.1 and B.2 were used as a precursor assessment to the final soil grading and classifications as per USCS and/or BS 5930 systems discussed in section 4.3.4.

Table 4.1: The ground water table depths from the project areas

Foundation		**Excavation Pit**		**Water table depth from GL**	**Foundation base below GL**
Location	**Type**	**Depth**	**Type**		
AP 108/15	Poor Soil	10m	Borehole	Nil (below 10m)	2.75m
AP 108/20	Good Soil	3m	Trial Pit	Nil (below 4m)	3.50m
KL 30	Waterlogged	10m	Borehole	1.140m	4.50m
AP 104/5	Pile	20m	Borehole	0.300m	~12.80m

Table 4.2: The summary of soil strata descriptions from the project areas

Foundation		***Insitu* soil strata Details (Soil Profile Table - Pits)**	**Soil Classifications (USCS & BS 5930 -Lab)**
Location	**Depth (m)**		
AP 108/15	2.45-4.2	Grey-dense clayey Sand	Silty SAND (SM)- USCS
AP 108/20	0.1-3.5	Brownish-orange laterite (***Murram****) with duricrust: *(medium-dense gravelly-Sand)	Clayey SAND with gravel (SC)- [USCS classification]
KL 30	4.5-6.5	Moist reddish brown, mottled grey, hard gravelly-Clays	Gravelly CLAYS of intermediate plasticity (CI) [BS 5930 classification]
AP 104/5	12.7-15.0	Slightly moist greyish brown, medium-dense clayey Sand	Clayey SAND (SC) [USCS classification]

4.2.2 Dynamic Probing Light

Dynamic Probing Light (DPL) tests were done to determine the blows per 10 cm penetration (N_{10}) readings, consistency descriptions, and computations of unit point (r_d) resistance, and dynamic point (q_d) resistance/soil bearing capacity as summarised in Table 4.3 below, with more details in Appendix B.3. The DPL's N_{10} readings of 10 to 54 under the respective penetration rates (e), corresponded to granular soils of medium-dense consistency especially coarse-grained sandy soils as shown in Table 2.8 in section 2.6.5 (BS EN 22476-2, 2005; Nilsson, 2012).

The DPL test results were used in the determination of the soil's preliminary consistency description, unit point resistance and dynamic point resistance (bearing capacity) as shown in in section 2.6.5.

Table 4.3: The DPL result summary for AP 108/20

Selective Depth (m)	M_1 (kg)	N_{10}	e	r_d (Mpa)	q_d (Mpa)
1.0	10.252	10	0.010	4.90	2.7
2.0	10.252	54	0.002	26.46	13.2
3.0	10.252	13	0.008	6.37	2.9
3.5	10.252	14	0.007	6.86	2.9

Where:
M_1 = mass of the hammer; and N_{10} = blows per 10 cm penetration
e = penetration rate (m per blow); and r_d = unit point resistance; q_d = dynamic point resistance

4.2.3 Standard Penetration Testing

The SPT test was done to determine the soil's consistency descriptions based on the N-values, and the parameters for bearing capacity calculations as summarised in Table 4.4, with more details found in Appendix B.4.

The SPT N-value of the fine-grained soils at KL 30 of 100, corresponded to a "hard" soil consistency description, whereas the N-values of the coarse-grained soils at AP 108/15 and AP 104/5 were 24 and 27 respectively, both corresponding to "medium-dense/compact" soil consistency descriptions according to Tables 2.5 and 2.5 in section 2.6.4. The SPT test results obtained, were thus, used in the determination of the soil's preliminary consistency descriptions based on the N-values (BS 5930, 1999) and parameters for bearing capacity analysis (Bowles, 1997; Das, 2016; Das and Sobhan, 2018).

Table 4.4: The SPT result summary for locations

Locations	SPT N-value	Corrected N_{55}	Soil consistency	Soil Classification (BS 5930 and USCS)
KL 30	100	77	Hard	Gravelly clays of intermediate plasticity (CI) - BS 5930
AP 108/15	24	19	Medium-dense	Silty Sand (SM) - USCS
AP 104/5	27	20	Medium-dense	Clayey Sand (SC) - USCS

4.2.4 Soil Resistivity Testing

The soil resistivity test was done to determine the corrosiveness of the soils based on the apparent resistivity values as summarised in Tables 4.5 and 4.6 below, with more details found in Appendix B.5. The resistivity values showed that the soils at locations KL 30, AP 108/20 and AP 104/5 were essentially non-corrosive, whereas the soil at AP 108/15 was highly corrosive.

The above soil resistivity results were used as a preliminary and non-conclusive test to provide generalised insitu environmental exposure conditions which may lead to steel depassivation and corrosion and affect the structural design of the reinforced concrete foundations. Thus, for a more conclusive study, chemical analysis-tests were deemed necessary regardless of the level of corrosiveness encountered as discussed in section 4.3.6 (Robinson, 1993; Roberge, 2000; BS EN 206, 2013; Das, 2016).

Table 4.5: The insitu soil resistivity result summaries

S/No	Locations	Average of soil resistivity (Ωm)	Soil corrosiveness description (Roberge, 2000)
1	KL 30	220.5	Essentially non-corrosive
2	AP 108/15	29.845	Highly corrosive
3	AP 108/20	1201.472	Essentially non-corrosive
4	AP 104/5	285.192	Essentially non-corrosive

Table 4.6: Soil resistivity explanation (Robinson, 1993; Roberge, 2000)

S/No	Soil Resistivity (Ω·m)	Soil Corrosiveness
1	Greater than 200	Essentially non-corrosive
2	100-200	Mildly corrosive
3	50-100	Moderately corrosive
4	30-50	Corrosive
5	10-30	Highly corrosive
6	Less than 10	Extremely corrosive

4.3 Laboratory Tests on soil samples

4.3.1 Specific gravity (Particle Density)

The specific gravity (Particle Density) test was done to provide a general preliminary description of the soil as a component of its index properties before conducting the particle size distribution/grading and soil classification tests, as summarised in Tables 4.7 and 4.8, with more details in Appendix B.6.

The obtained specific gravity values showed the soils at location AP 108/15 to be sand with silty particles, AP 108/20 to be gravelly soil with clay mineral compositions, KL 30 to be clay with gravel particles, and AP 104/5 to be sand with clay compositions. These descriptions are comparable but inconclusive to the final results as per the Unified Soil Classification System (USCS) and/or BS 5930 soil classification system as discussed in section 4.3.4. For that reason, higher specific gravity values give higher load carrying capacities and thus, more strength for foundation soils since an increase in specific gravity increases the shear strength parameters and the soil's suitability as a construction material (Tuncer and Lohnes, 1977; Roy and Dass, 2014; Surendra and Sanjeev, 2017).

Table 4.7: Summary of specific gravity values on site

S/No	**Location**	**G_s Range (Mg/m^3)**	**Type of soil (Generalised)**	**Soil Classifications (USCS & BS 5390)**
1	AP 108/15	2.370-2.777	Sand with silty particles	Silty sand (SM)- USCS
2	AP 108/20	2.380-2.483	Gravelly soil with clay mineral compositions	Clayey sand with gravel (SC)- USCS
3	KL 30	2.453-2.739	Clay with gravel particles	Gravelly clays of intermediate plasticity (CI)- [BS 5930]
4	AP 104/5	2.64-2.73	Sand with clay composition	Clayey sand (SC)- USCS

Table 4.8: Specific gravities of some soils (Das, 2016; Das and Sobhan, 2018)

S/No	**Type of Soil**	**Specific Gravity (G_s) range (Mg/m^3)**
1	Gravel	2.65 - 2.68
2	Quartz Sands	2.64 - 2.66
3	Silty	2.67 - 2.73
4	Clay	2.70 - 2.90
5	Chalk	2.60 - 2.75
6	Loess	2.65 - 2.73
7	Peat/Organic soils	1.30 - 1.90 (Less than 2.0)
Clay soil mineral compositions		
8	K-Feldspars (1)	2.54 - 2.57
9	Montmorillonite (2)	2.35 - 2.70
10	Illite (2)	2.6 - 3.0
11	Kaolinite (2)	2.6 - 2.68
12	Biotite (1)	2.8 - 3.2

References: (1) Lambe and Whitman, 1969; (2) Mitchell, 1993

4.3.2 Direct Shear Test

The direct shear test was done to determine the soil's cohesion (c) and the angle of internal friction (ϕ) used in the computation of the soil's bearing capacities as summarised in Table 4.9, with more details in Appendix B.8.

The value of friction angle (ϕ) was observed to be greater than cohesion (c) $(i.e\ \phi > \emptyset)$ at 5.2m depth, and friction angle (ϕ) less than cohesion (c) $(i.e\ \phi < \emptyset)$ at 10.4m depth, despite both soil strata being classified as clayey-sand soils. This difference is indicative of an increased clay and silt composition in the bottom soil strata because they induce the sand with increased interlocking behaviour/cohesion (Smith, 2014; Das, 2019).

The obtained results of cohesion (c) and friction angle (ϕ) were used in the computation of the soil's bearing capacities, the pile skin and end-bearing resistances using the lateral earth pressure coefficient as discussed in sections 2.6.6, 2.6.7, and 3.7.2.

Table 4.9: Direct shear summary results for AP 104/5

S/No	Depth	Width	Clay and silt (%) from PSD*	Bulk Density	Cohesion (c)	Friction angle (ϕ)	Bearing capacity (q_{all})
	m	m		Mg/m^3	kPa	°	kPa
1	5.20	1	43.8%	1.790	12.3	21	342
2	10.40	1	63.2%	1.745	18.5	15	348

Note: * PSD: Particle Size Distribution or soil grading

4.3.3 Consolidation Tests

The one-dimensional consolidation test was done to determine the coefficients of consolidation (C_v) and volume compressibility (m_v) used in the subsequent explanations of the probable volume levels of compressibility (compression index) and a generalised description of the possible underlying soil strata type as summarised in Tables 4.10 to 4.14, with more details in Appendix B.8.

The obtained consolidation C_v value of 0.0042 cm^2/s in the range of 0.00032 to 0.0032, corresponding to a medium category of consolidation, typical of 15-25% clays of the low plastic clay (CL) USCS classification type; whereas, the m_v values of 0.187 m^2/MN in the range of 0.25-0.125 (Table 4.12) and/or 0.1-0.3 (Table 4.13) corresponded to stiff or firm clays of consolidated lake deposits/lacustrine/swampy soils having medium compressibility properties of 0.05 to 0.15 compression index (C_c) (Smith, 2014; Carter and Bentley, 2016).

Table 4.10: Consolidation Test Results for AP 104/5

Test Depth (m)	(e_o)	(γ_b)	(c_v)	(m_v)	(p_o)
	(-)	(Mg/m^3)	(cm^2/s)	(m^2/MN)	(kPa)
10.4 - 10.7	0.752	1.745	0.0042	0.187	201

Where:
e_o = Initial void ratio; γ_b = Initial bulk density; c_v = Coefficient of consolidation
m_v = Coefficient of volume compressibility; p_o = Pre-consolidation pressure

Table 4.11 Coefficients of consolidation (George *et al*., 2006; Carter, 2016*)

Range of C_v (cm^2/s)	Category	Typical material	Soil classification (USCS)
< 0.000032	Very Low	-	-
0.000032-0.00032	Low	>25% Clay	Medium plasticity clays (CL-CH), and Volcanic silt (MH)
0.00032-0.0032	Medium	15-25% Clay	Low plasticity clay/mud (CL)
0.0032-0.032	High	<15% Silt	Organic silt (OL)
> 0.032	Very High	-	-

Note: $1\ m^2/year = (5/15768)\ cm^2/s$; and C_v = coefficient of consolidation
***** Carter and Bentley, 2016 **[**Adapted from Holtz and Kovacs (1981)]

Table 4.12: Ranges of coefficient of vol. compressibility ($\boldsymbol{m_v}$) (Smith, 2014)

S/No	Soil type	$\boldsymbol{m_v}$ (m^2/MN)
1	Peat	10.0 - 2.0
2	Plastic clay (normally consolidated alluvial clays)	2.0 - 0.25
3	Stiff clay	0.25 - 0.125
4	Hard clay (boulder clays)	0.125 - 0.0625

Table 4.13: Coefficient of vol. compressibility (Carter and Bentley, 2016)

M_v m^2/MN	C_c	Compressibility category	Type of soil material indicated
<0.05	0.025	Very Low compressibility	Hard, over-consolidated glacial till, stiff weathered rocks and hard clays
0.05-0.1	0.025-0.05	Low compressibility	Stiff Glacial Till (Boulder Clay), marls, very stiff tropical residual clays

M_v m^2/MN	C_c	Compressibility category	Type of soil material indicated
0.1-0.3	0.05-0.15	Medium compressibility	Firm clays of consolidated swampy/lake deposit/lacustrine soils, glacial outwash clays, weathered marls, firm glacial till, normally consolidated clays at depth, firm tropical residual clays
0.3-1.5	0.15-0.75	High compressibility	Poorly consolidated alluvial clays such as estuarine deposits, and sensitive clays
>1.5	0.75-5+	Very High compressibility	Highly organic alluvial clays and peats

Where:
C_c = compression index; m_v = coef. of vol. compressibility; a_v = coefficient of compressibility

$$m_v = \left(\frac{a_v}{1+e_o}\right) = \left[\left(\frac{\delta_e}{\delta_p}\right) x \frac{1000}{(1+e_o)}\right] \text{(Adopted from Bowles, 1997)}$$

4.3.4 Particle Size Distribution (Soil grading)

The soil grading test was done to determine the distribution of the different particle-sizes in the soil mass, and the gradation of the soil as summarised in Table 4.14, with more details found in Appendix B.6.

The particle size analysis showed that the soils at the formation levels of the locations AP 108/15 were of poorly graded silty sand (SM), AP 108/20 were well graded clayey sand with gravel (SC), AP 104/5 were poorly graded clayey sand (SC) using the USCS classification system; whereas, the soils at KL 30 were gap graded gravelly clay (CI) as per the BS 5930 classification system.

The above gradation and soil classification results were used to finally confirm the previously inconclusive descriptions from the Test trial pit and/or borehole soil strata, parts of the preliminary soil consistency descriptions using DPL, specific gravity soil generalisations, material type identifications using the consolidation's m_v and C_v values, and the general soil type descriptions under plasticity index (PI) interpretations as discussed in sections 4.2.1, 4.2.2, 4.3.1, 4.3.3, and 4.3.5 respectively (Smith, 2014; Das, 2016; Das and Sobhan, 2018).

Table 4.14: Summary of grading of locations

S/No	location	Soil Grading	Formation Level	Soil Classification
1	AP 108/15	Poorly graded	2.75m	SM (USCS)
2	AP 108/20	Well-graded	3.50m	SC (USCS)
3	AP 104/5	Poorly graded	~12.80m	SC (USCS)
4	KL 30	Gap-graded	4.50m	CI (BS 5930)

4.3.5 Plasticity Index Interpretations

The plasticity index (PI) values derived from the Atterberg limit tests, were done to classify the cohesiveness and swell potentials of fine-grained soils in the general context when correlated with other soil properties, as summarised in Tables 4.15 to 4.17 below, with more details found in Appendix B.6.

The Atterberg test showed that the soils at location AP 108/15 had PI values in the range of 7-17, corresponding to medium-plastic soils of cohesive silty-sand type, whereas AP 108/20, KL 30 and AP 104/5 had soils with PI values greater than 17 (>17), corresponding to high plastic soils of cohesive clay type. Meanwhile, all the above locations had Liquid Limit (LL) values less than 50 (<50), corresponding to fine-grained soils with low swell potentials.

The above results of PI and LL interpretations were used in complementing the final classification and grading descriptions of the fine-grained soils as discussed in sections 4.3.4 (Das and Sobhan, 2018; Das, 2019).

Table 4.15: Atterberg limit summaries for the sites at the formation level

S/No	Location	Atterberg Limits			Formation Level	PI value range (From Table 4.16)
		LL	PL	PI		
1	AP 108/15	24.7	12.5	12.20	2.75m	7 - 17
2	AP 108/20	44.8	21.4	23.4	3.50m	> 17
3	KL 30	38.4	16.6	21.8	4.50m	> 17
4	AP 104/5	34.5	14.8	19.7	~12.80m	> 17

Table 4.16: PI interpretations and cohesiveness (Surendra and Sanjeev, 2017)

S/No	PI	Degree of Plasticity	Degree of Cohesiveness	Soil Type
1	0	Non-Plastic	Non-cohesive	Sand
2	< 7	Low Plastic	Partly cohesive	Silt
3	7-17	Medium Plastic	Cohesive	Silty-Sand
4	> 17	High Plastic	Cohesive	Clay

Table 4.17: Atterberg Limits & Swell Potential (Pitts, 1984; Kalantari, 1991)

S/No.	Liquid Limit (LL)	Plasticity Index (PI)	Swell Potential (SP)
1	< 50	< 25	Low
2	50 - 60	25 - 35	Marginal
3	> 60	> 35	High

4.3.6 Chemical Analysis Tests

The chemical tests were done as a more conclusive test following the preliminary soil resistivity test to determine the presence of the corrosion-causing chemicals of sulphates and/or chlorides, and pH values so as to describe the locations' environmental exposure conditions as summarised in the Tables 4.18 to 4.20 below, with more details found in Appendix B.7.

The SO_4 results showed that the soils at locations AP 108/15, AP 108/20 and AP 104/5 had values ≤ 3000 mg/kg (≤ 0.3% by weight) corresponding to XA1 exposure condition of slightly aggressive chemical environments; whereas, the KL 30 ground water had values of SO_4 > 600 ppm, corresponding to XA2 exposure condition of moderately aggressive chemical environment.

The pH values for AP 108/15 and AP 108/20 were 5.38 and 5.22 respectively, corresponding to XA2 exposure condition of moderately aggressive chemical environment, whereas KL 30 and AP 104/5 had pH values of 6.27 and 6.10 respectively, corresponding to XA1 exposure condition of slightly aggressive chemical environments. Hence, the chemical analysis and pH environmental conditions were reconciled to provide XA2 exposure conditions for all locations based on “worst-case scenario” approach (BS 4027, 1996; Stark, 2002; Michael *et al*., 2005; BRE-SD-1, 2005; BS EN 206, 2013).

The chemical analysis and pH test values were used to conclusively determine the corrosion-causing sulphates, chlorides and pH conditions following the preliminary soil resistivity tests as discussed in section 4.2.4.

In order to inhibit the effects of chlorides to form insoluble chloro-aluminates (C_3A) upon combining with Tricalcium Aluminate ($3CaO{\cdot}Al_2O_3$) in concrete, Sulphate Resistant Cements (SRC) of strength class 42.5 N and a 3.5% limited C_3A content are used under moderate water-cement ratios of 0.40 to 0.50 as illustrated in Figure 4.1 (Stark, 2002).

Table 4.18: Insitu chemical test results at the formation level

S/No	Location	Sulphate content (% by weight)	Chloride content (g/l)	pH value	Sample type
1	AP 108/15	0.05% (500 ppm)	0.007 (7 ppm)	5.38	Soil
2	AP 108/20	0.06% (600 ppm)	0009 (9 ppm)	5.22	Soil
3	KL 30	0.0686% (686 ppm)	10 (10,000 ppm)	6.27	Ground water
4	AP 104/5	0	0.021(21 ppm)	6.10	Soil

Where: 1 g/L = 1000 ppm; and 1 ppm = 1 mg/L = 0.001 g/L

Table 4.19: Measured results interpretations for sulphates

S/No	Location	Measured SO_4 values (ppm)	BS EN 206 (2013) SO_4 values (ppm)	Exposure condition (BS EN 206, 2013)
1	AP 108/15	500 ppm	≥ 2000 and ≤ 3000 (soil)	XA1
2	AP 108/20	600 ppm	≥ 2000 and ≤ 3000 (soil)	XA1
3	KL 30	686 ppm	> 600 and ≤ 3000 (water)	XA2
4	AP 104/5	0	≥ 2000 and ≤ 3000 (soil)	XA1

Table 4.20: Limiting values for chemical exposure (BS EN 206: 2013)

Chemical characteristic	**Exposure Risk Conditions (BS EN 206: 2013)**		
	XA1: Slightly aggressive chemical environment	**XA2:** Moderately aggressive chemical environment	**XA3:** Highly aggressive chemical environment
Ground Water			
SO_4^{2-} (mg/l) *1 ppm = 1 mg/l	≥ 200 and ≤ 600	> 600 and ≤ 3000	> 3000 and ≤ 6000
pH	≤ 6.5 and ≥ 5.5 (5.5 - 6.5)	< 5.5 and ≥ 4.5 (5.5 - 4.5)	< 4.5 and ≥ 4.0 (4.5 - 4.0)
Soil			
SO_4^{2-} (mg/kg)	≥ 2000 and $\leq 3000^{*}$ ($\geq 0.2\%$ and $\leq 0.3\%$)	$> 3000^{*}$ and ≤ 12000 ($> 0.3\%$ and- $\leq 1.2\%$)	> 12000 and ≤ 24000 ($> 1.2\%$ and $\leq 2.4\%$)

NB:

[*] The 3000 mg/kg limit is reduced to 2000 mg/kg, where there is sulphate ion accumulation risk in the concrete due to drying and wetting cycles/ capillary suction.

- 1 mg/kg = 0.0001% by weight; 1% by weight = 10000 mg/kg
- 1 g/L = 1000 ppm; and 1 ppm = 0.001 g/L = 1 mg/L

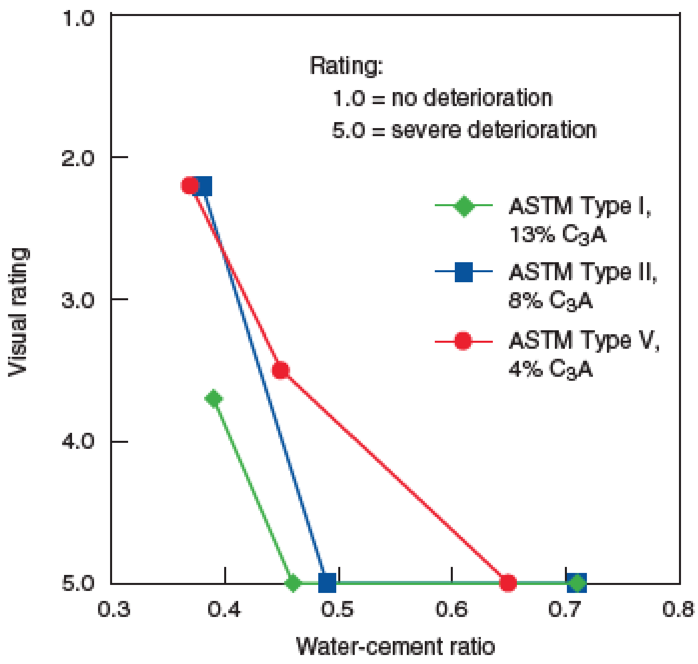


Figure 4.1: Average 16-year ratings of concrete in sulphate soils (Stark 2002).

4.4 Concrete cube compressive strength tests

The concrete cube compressive strength test was done in conformity to BS EN 12390-1: 2012 and BS EN 12390-2: 2009, to determine the 7-day and 28-day strengths as a confirmatory quality control test of the 25 MPa design concrete strength using the 42.5N Sulphate Resistant Cement (SRC), as shown in Figure 4.2 and summarised in Table 4.21 below, with more details in Appendix C.1.

The test results showed that all the locations had 7-day test strength values in the 104.2 - 123.36% range of the 25 MPa design value, and 28-day test strength values in the range of 155.12 - 211.08% of the 25 MPa design value.

The compressive test values above were used to confirm and provide assurance to the foundation's design concrete strength value of 25 MPa using 42.5N Sulphate Resistant Cement as a remedy to the sulphate and chloride attacks, as discussed in section 4.3.6 (BS EN 12390-2, 2009; BS EN 12390-1, 2012).

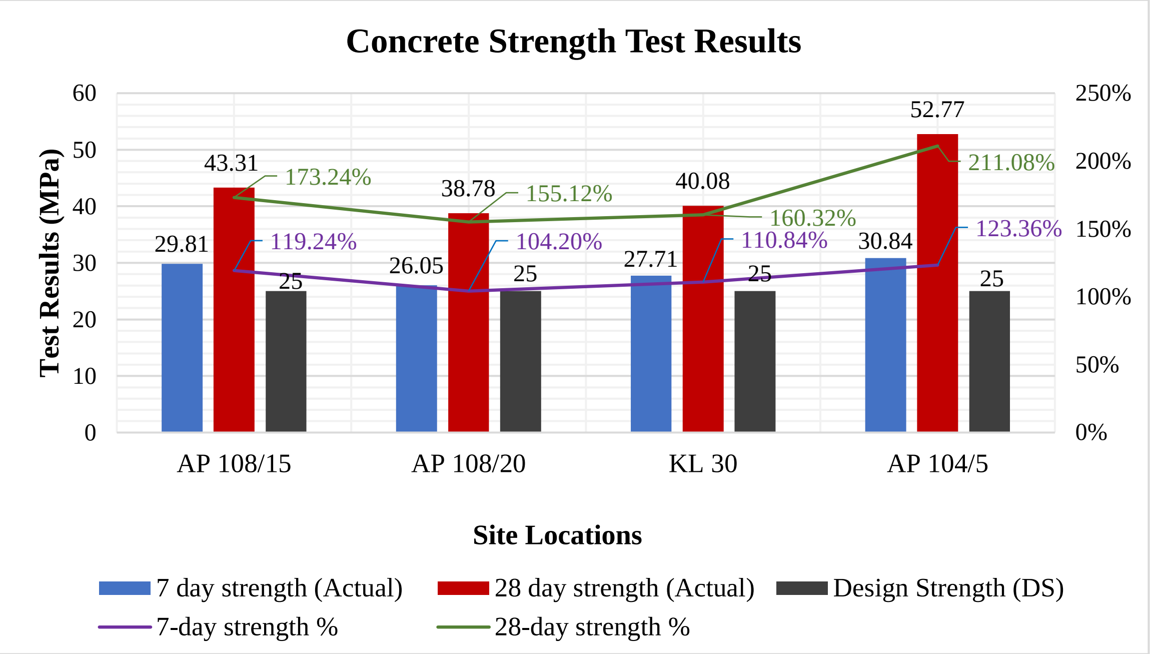


Figure 4.2: Compressive Concrete cube strength results

Table 4.21: Compressive Concrete cube strength result summaries

Location	Tested Cube strength values (For a 28-day Design Strength of 25 MPa)			
	7-day strength		28-day strength	
	Results (MPa)	% of the 25Mpa Design Strength	Results (MPa)	% of the 25Mpa Design Strength
KL 30	27.71	110.84%	40.08	160.32%
AP 108/15	29.81	119.24%	43.31	173.24%
AP 108/20	26.05	104.20%	38.78	155.12%
AP 104/5	30.84	123.36%	52.77	211.08%

4.5 Static Load Tests

The insitu static load tests were done to determine the insitu displacement and load capacity values of the foundations when under tension/uplift, compression, and lateral load test methods in conformity to IEC 61773 (1996) and/or COMESA/FDHS 293 (2007) as summarised in Tables 4.22 to 4.24, with more details found in Appendix D.

The static load test results showed that the location AP 104/5 exhibited maximum displacement values of 0.09 mm, -0.83 mm and 2.39 mm under tension, compression and lateral load tests respectively; whereas locations AP 108/15, AP 108/20 and KL 30 exhibited maximum tension displacement values of 0.83 mm, 0.19 mm and 4.74 mm respectively against limiting reference displacement values of 25 mm for both tension and compression load tests, and a 50 mm limiting reference displacement value for the lateral load test (IEC 61773, 1996; COMESA/FDHS 293, 2007).

The above static load test results were used in determining the slope (C_1) of the hyperbolic model graph's empirical line equation, and calculation of the insitu foundation's load capacity (R_c) using the Chin-Kondner extrapolation (1971) as discussed in section 3.11. The actual insitu load capacities of the test-foundations under static load methods were 105.29% to 249.14% fraction of the prescriptive design values, which reaffirmed the conclusion that the load capacity results of the insitu-tested full-scale foundations exceeded the prescriptively designed load capacity values, as shown in Table 4.24.

Table 4.22: Insitu static load test summaries

Location	Maximum Displacements (mm)			IEC 61773 (1996) Displacement Limits		
	Tension (T)	Compression (C)	Lateral (L)	T (mm)	C (mm)	L (mm)
AP 104/5	0.09	-0.83	2.39	25	25	50
AP 108/15	0.83	-	-			
AP 108/20	0.19	-	-			
KL 30	4.74	-	-			

Table 4.23: Slope Readings for insitu static load tests

Foundation	Graph Line Slopes (x 10^{-3})		
	Tension Test	Compression Test	Lateral Test
AP 104/5	0.7223	0.5019	5.2591
AP 108/15	0.9996	-	-
AP 108/20	1.3568	-	-
KL 30	0.8755	-	-

Table 4.24: Foundation load capacity summaries for the locations

Location	Load Capacities (kN)					
	Tension Test		Compression Test		Lateral Test	
	Insitu load	Ultimate Design Load	Insitu load	Ultimate Design Load	Insitu load	Ultimate Design Load
AP 104/5	1384.47	555.69	1992.43	1077.08	190.15	180.59
AP 108/15	1000.40	945.36	-	-	-	-
AP 108/20	737.03	594.45	-	-	-	-
KL 30	1142.20	962.30	-	-	-	-

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

In the last couple of years, Static load tests for foundations has been applied in several new projects in Uganda, and one such area is the inclined foundation columns/chimneys of the overhead power transmission lines.

The main objective of this study was to conduct a comparative analysis of the foundation's performance with respect to its load-displacement response using the prescriptive design and insitu static load test methods under sustained axial loading conditions in order to validate the obtained allowable and/or ultimate load bearing capacities and the foundation's design assumptions used. This was achieved by studying the constitutive behaviour of the soils and the findings of the static load tests on the composite foundation structure.

Considerations of the aforementioned transmission line specifications limited the 'step-pad' foundations to only uplift/tension load tests as compared to the pile foundation where all uplift, compression and lateral tests were conducted. These load 'testing-type' limitations were necessary to allow for a factual and manageable project. The knowledge and insight gained and calculation procedures, however, are not limited to the materials and configurations used in the research study.

5.2 Conclusions

The following conclusions were drawn:

5.2.1 Insitu Load Capacities

The insitu load capacities of the test foundations under static load method were 105.29% to 249.14% fraction of the prescriptive load capacities values. Hence, the static load test confirmed that the prescriptive design approaches use equations and methods governing a linear-elastic boundary value problem in the design value extrapolations instead of the more-realistic plastic and non-linear approach.

5.2.2 Maximum displacement values

The maximum displacement values from the static load tests differed considerably from that of the prescriptive design and technical specifications by being about 0.36% to 18.96% fraction of the 25 mm prescriptive limit under uplift and 3.32% fraction of the prescriptive 25 mm under compression and 4.78% fraction of the prescriptive 50 mm limit under lateral test. The static load test displacements were less than 20% of the prescriptive/theoretical values, thus, reducing displacement overdesigns by 80%. The prescriptive values were 80% higher due to multiple design assumptions and higher factors of safety to minimise human error on site.

5.2.3 Compressive strength of test foundation concrete

Due to the acidic nature of the soils or ground water, and the technical specification’s proposal to use 42.5N Sulphate Resistant Cement (SRC), this created an overdesign in the compressive strength of concrete ranging from 104.2% to 123.36% of the design compressive strength at 7 days, and a 155.12% to 211.08% at 28 days, due to lack of a low grade 32.5N SRC in Uganda.

5.3 Recommendations

The following recommendations have been made:

5.3.1 Current Research Issues

Although this study has advanced the current state of understanding of the functioning of static load tests, it has been limited in its scope due time and financial constraints, and further research needs to be done in the areas that fall outside the scope of this project. The most important of the areas that need further research is the influence of temperature variations and the rate of backfill soil compaction on the ultimate loading bearing capacity readings.

5.3.2 Company (Sinohydro Corporation, KPTL, and UETCL)

The company needs to create a centralised database where all design data are collected, analysed and recorded for all projects as shown by the need for site-specific design-data in this research, and liaise with UIPE/ERB.

5.3.3 Kyambogo University

It is hoped that in future, the University could partially or fully fund researches of graduate and undergraduate students; and regularly update the centralised research database so as to mitigate duplication by students.

5.4 Future Research

Future research should be done in quantifying the influence of ambient temperature and/or weather variations, and the rate of insitu backfill-soil compaction on the results of the static loading tests.

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APPENDICES

Appendix A.1 - Site Location Maps

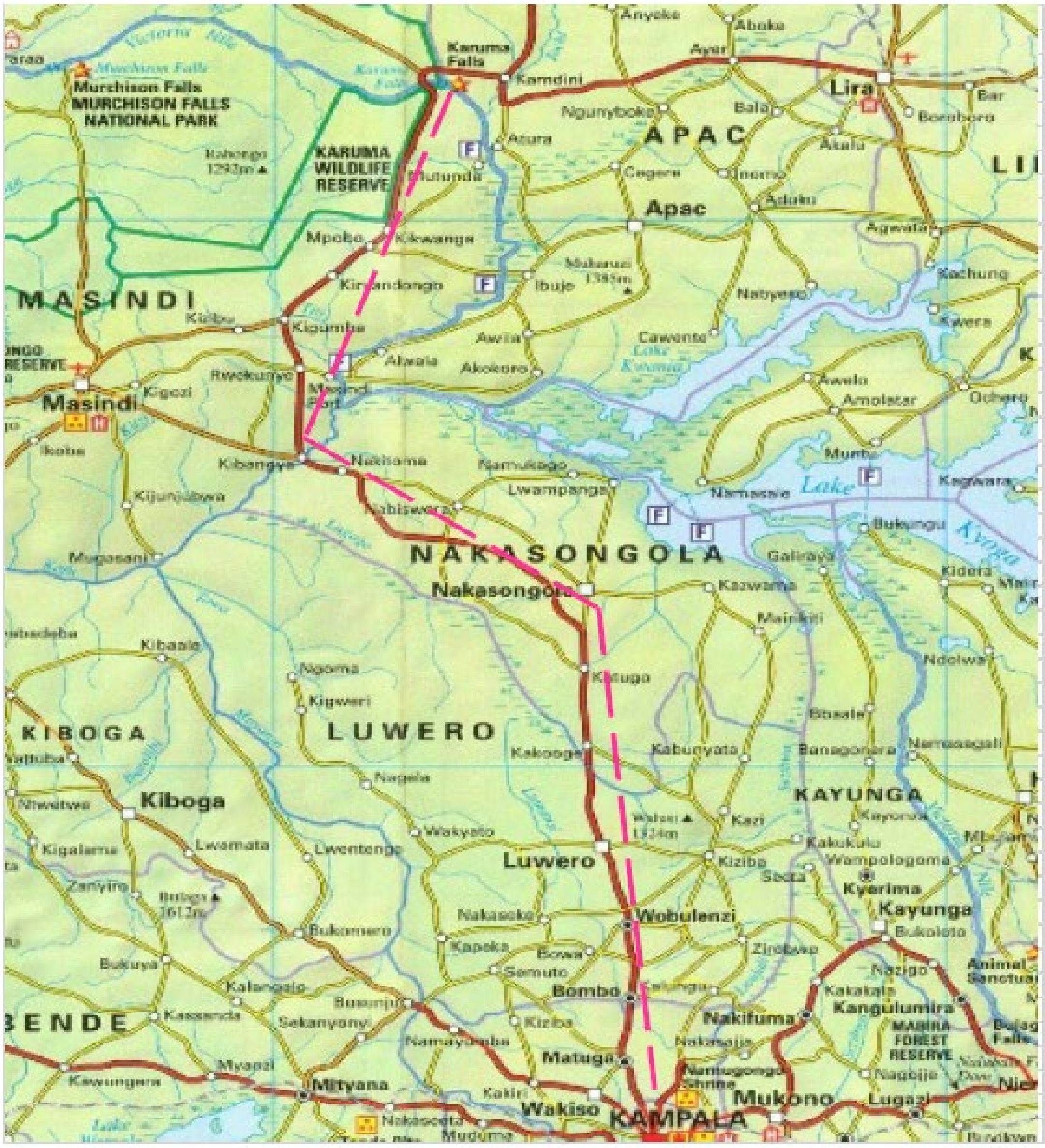


Figure A.1-1: 400kV Karuma-Kawanda Transmission Line Route (KIP, 2019)

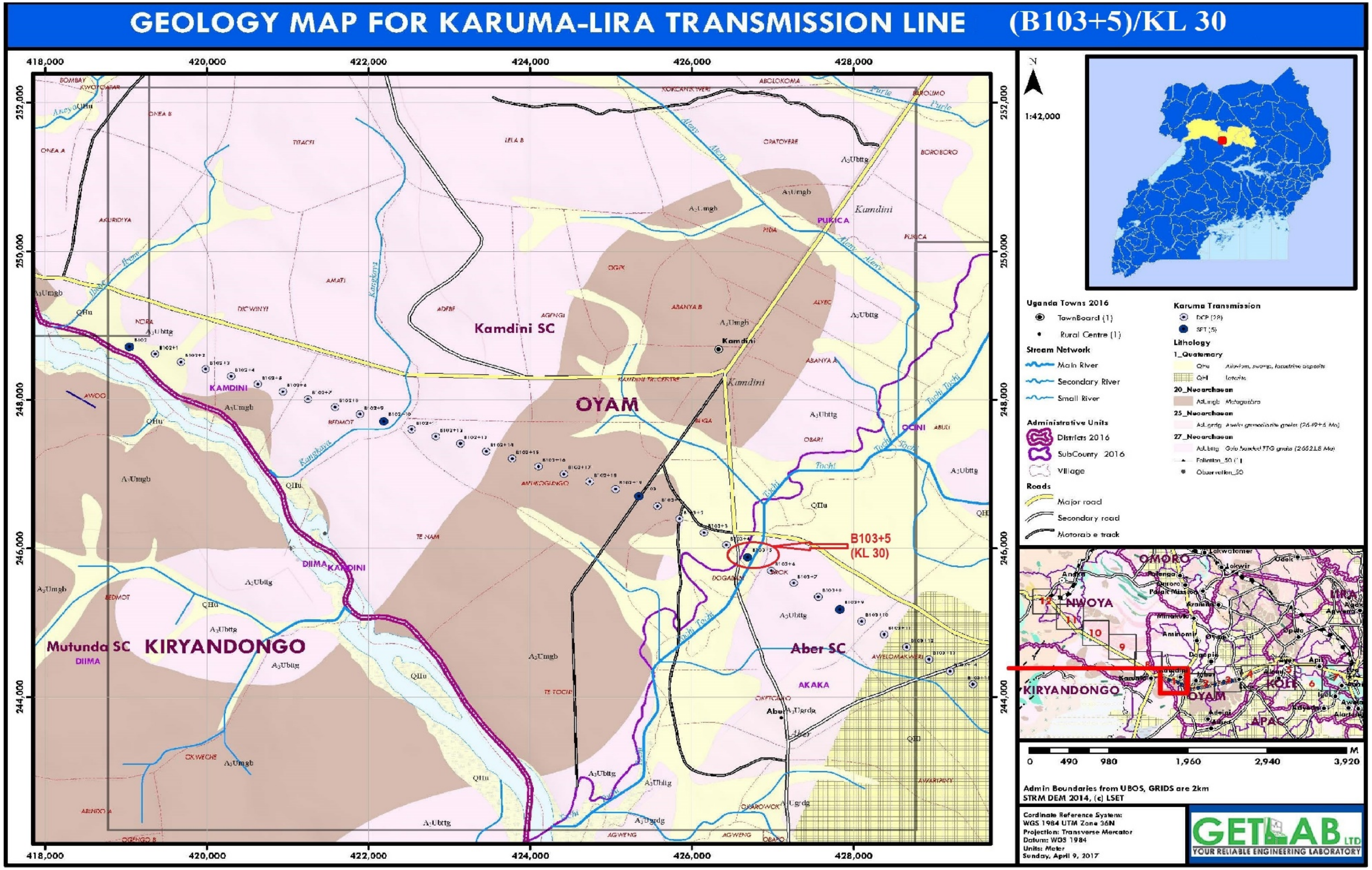


Figure A.1-2: Location Map for KL 30 (Getlab, 2019)

Appendix A.2 - Geology Map of Uganda

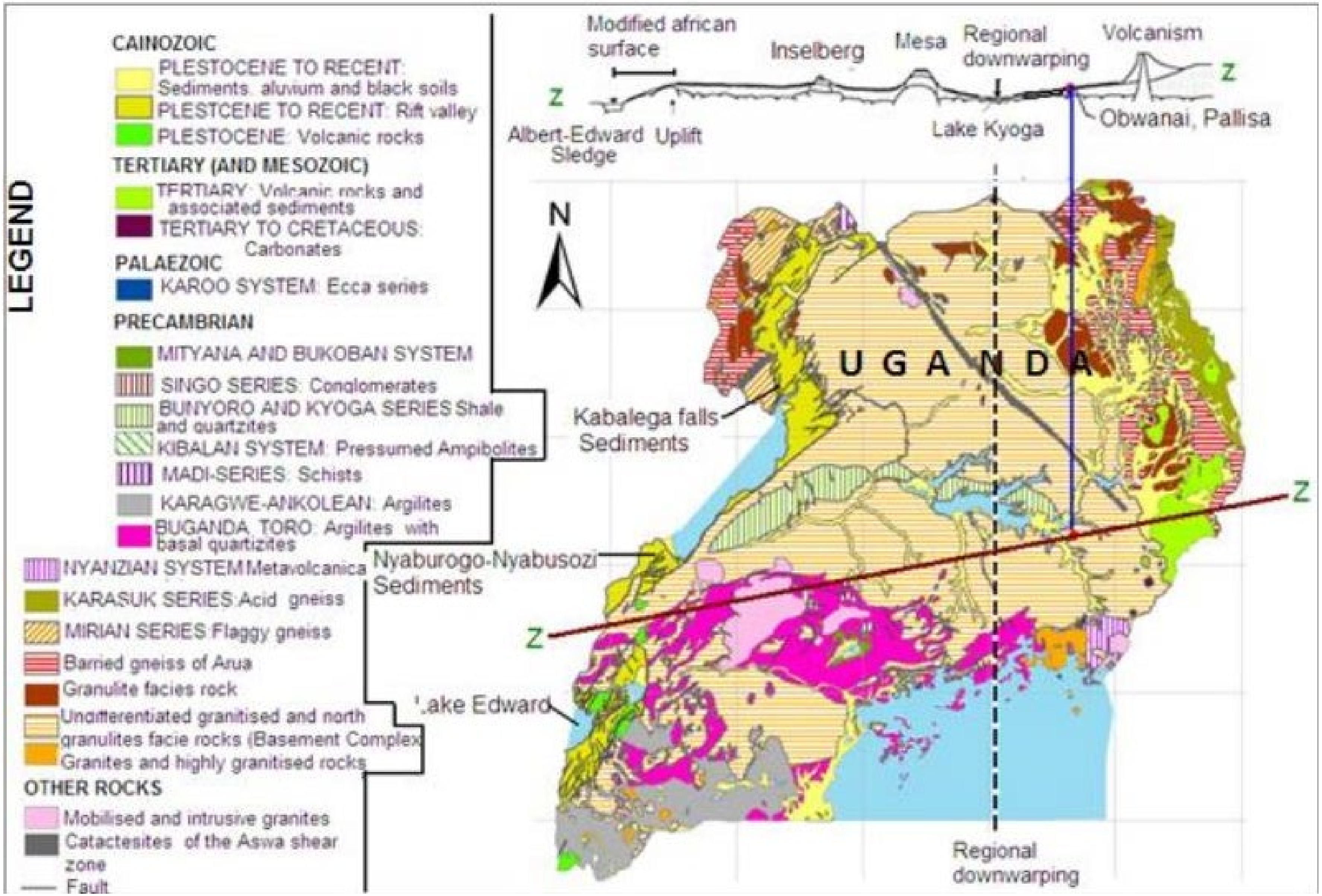


Figure A.2-1: The Late Quaternary Landscape of Uganda (DHI and COWI, 2011)

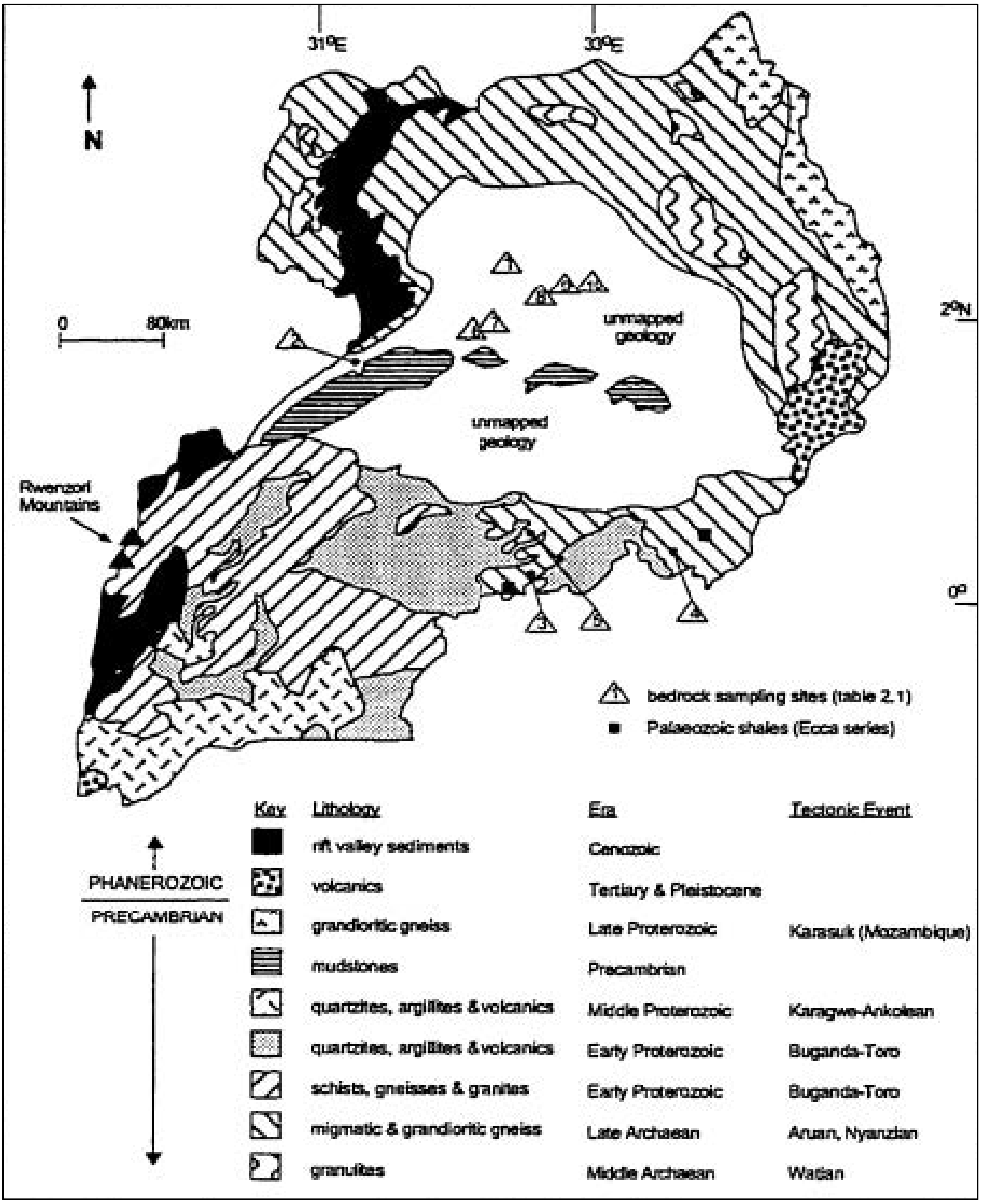


Figure A.2-2: The Geological Map of Albertine Graben (Geology and Mines, 1998)

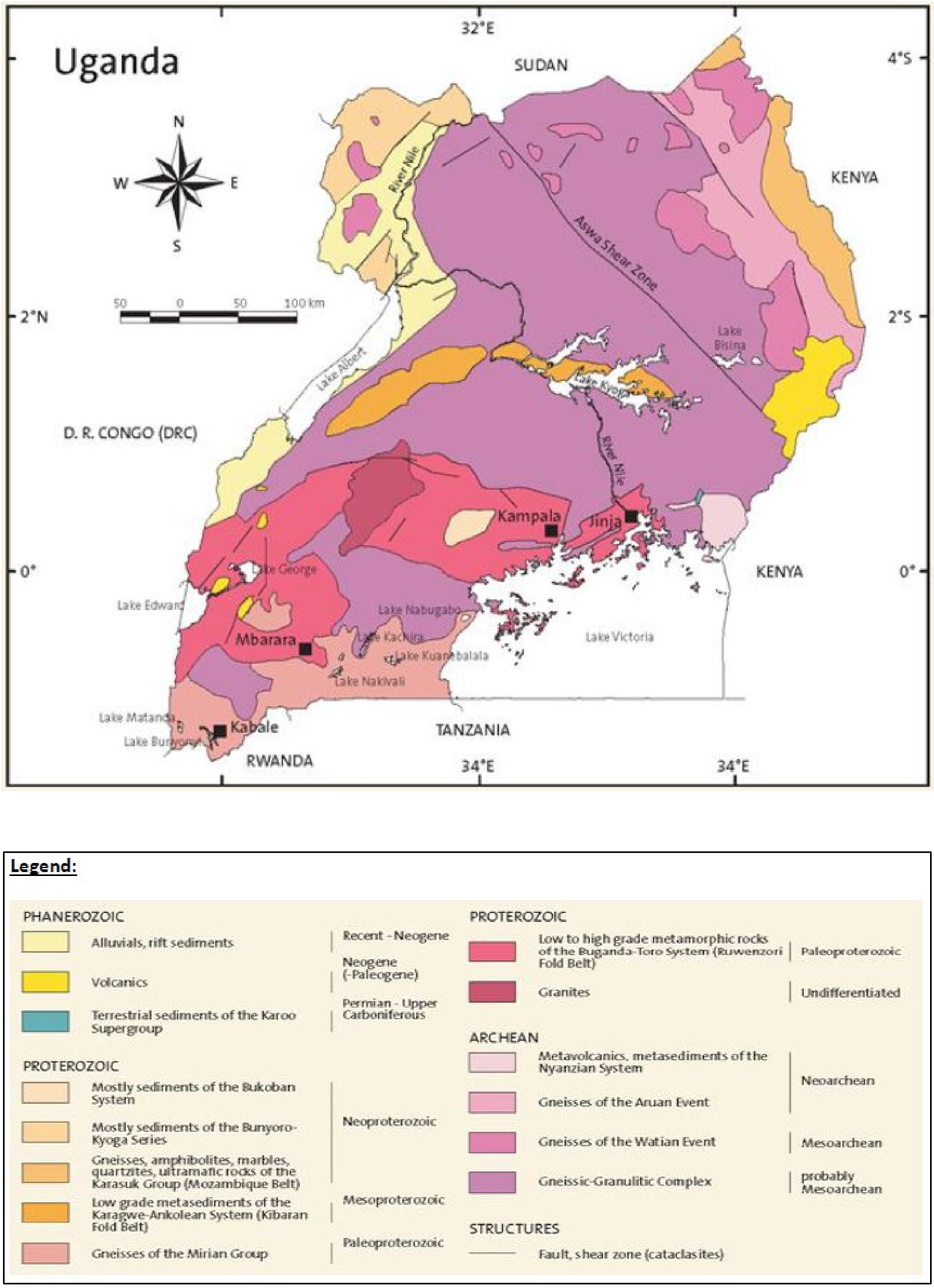


Figure A.2-3: Geology Map of Uganda (Macdonald, 1966 & Muwanga et al., 2001)

Appendix A.3 - The Seismic Map of Uganda

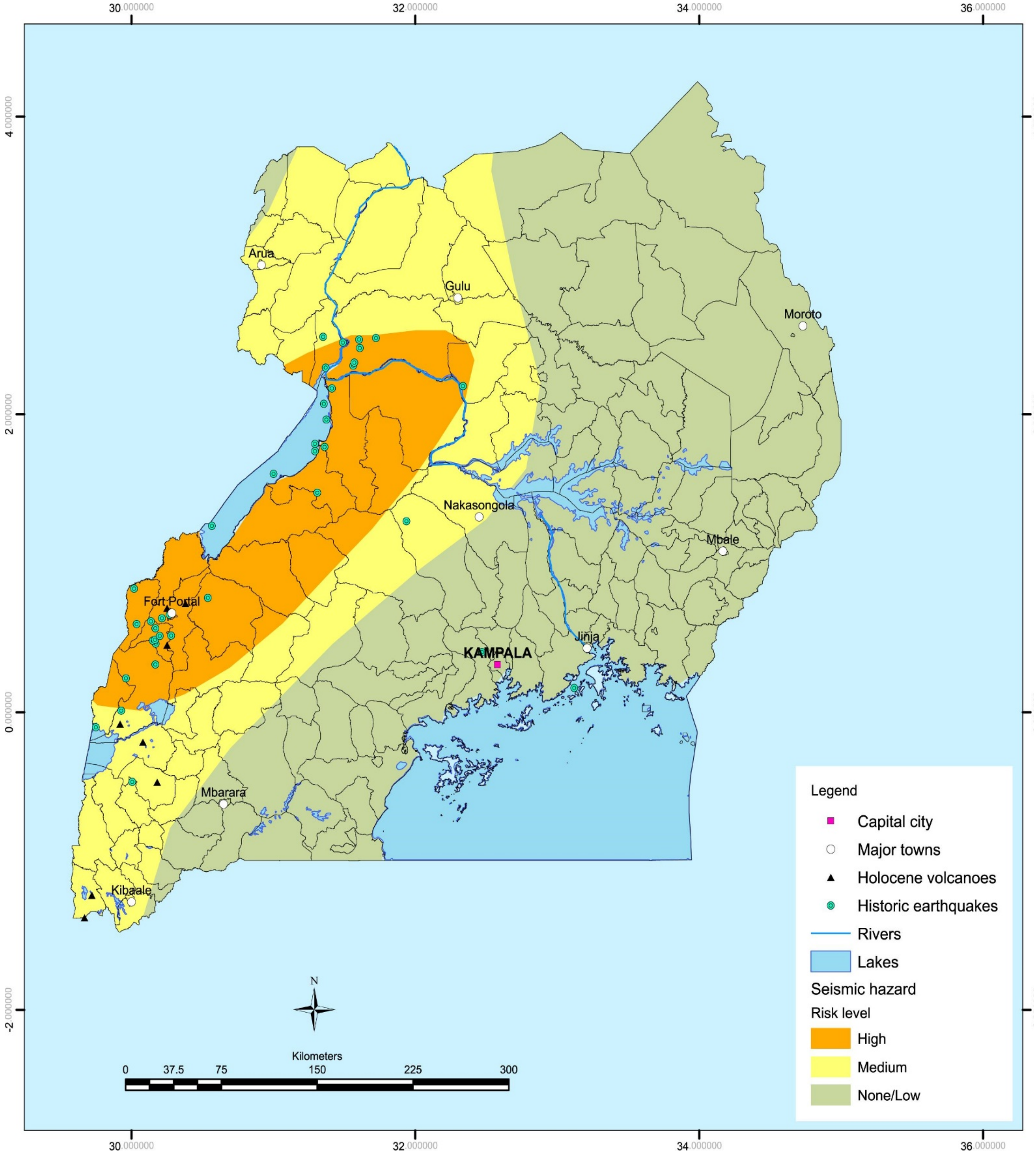


Figure A.3-1: The Seismic Map of Uganda (Geology and Mines, 2002)

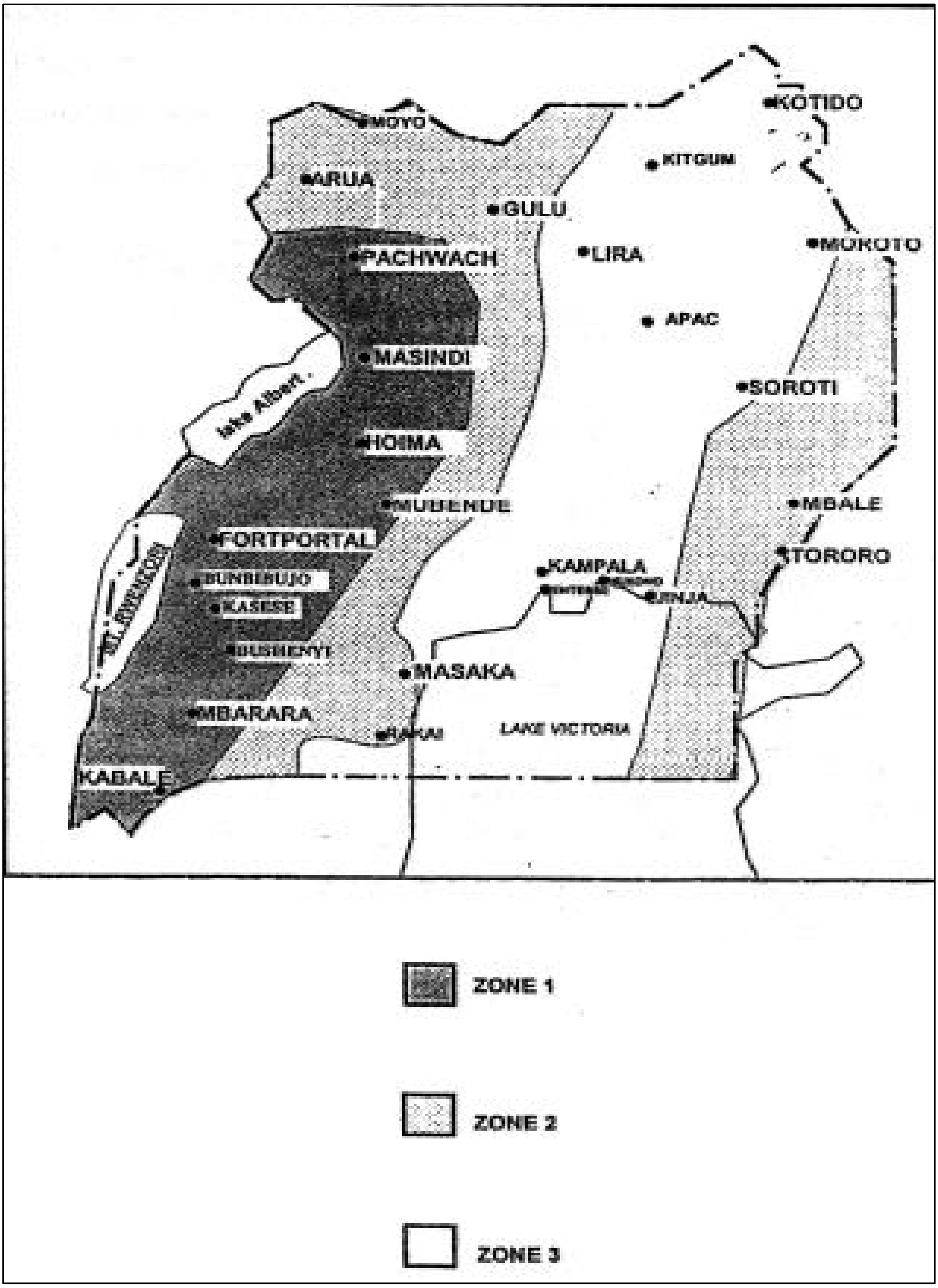


Figure A.3-2: Seismic Map of Uganda (MoWT, 2010; US 319, 2003)

Appendix A.4 - The Demarcated Rainfall Zones of Uganda

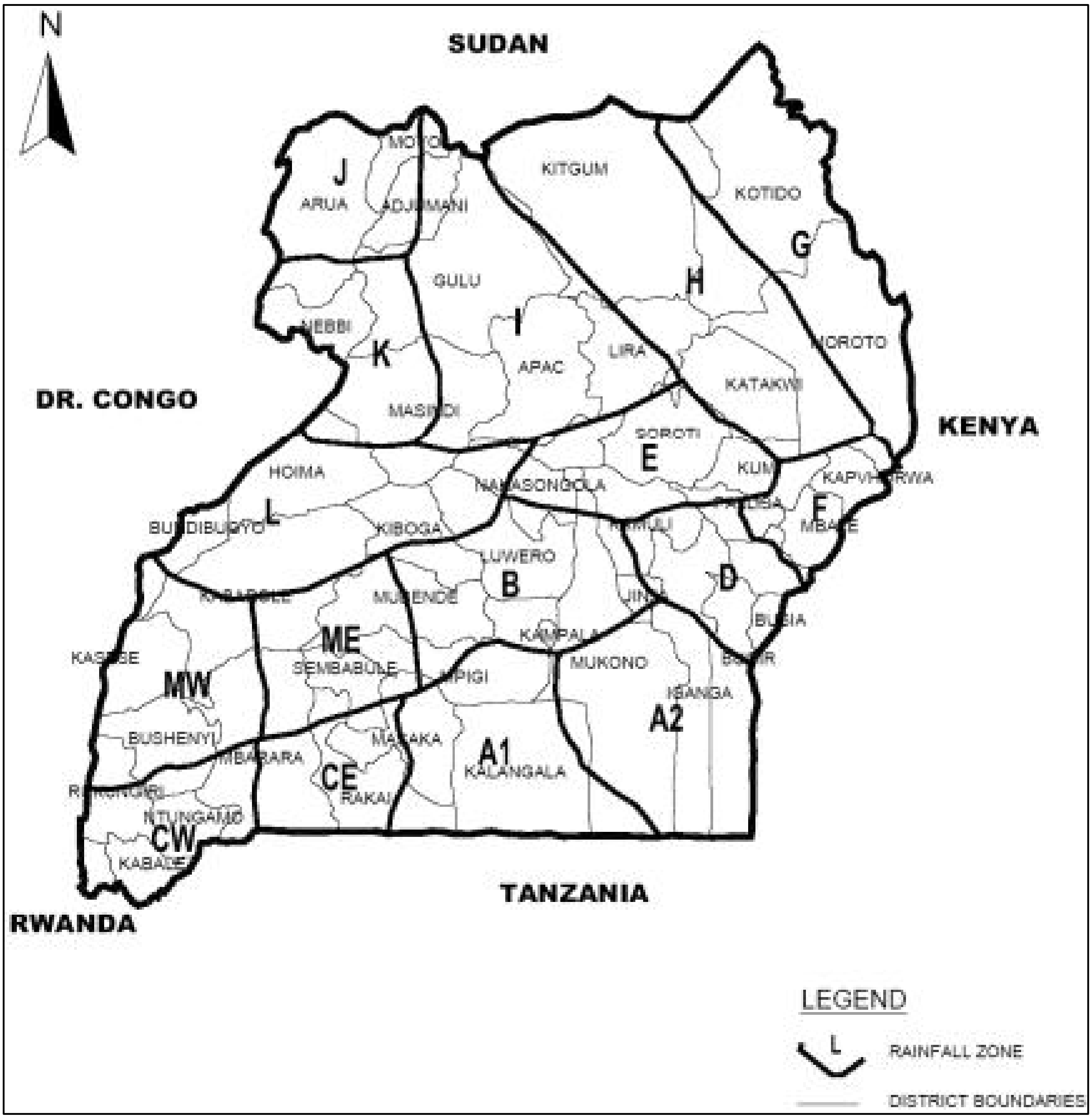


Figure A.4-1: Demarcated Rainfall Zones of Uganda (MoWT, 2010)

Appendix B.1 - Soil Profiles

Table B.1-1: Soil Profile Report for KL 30 (B103+5)

SOIL PROFILE REPORT

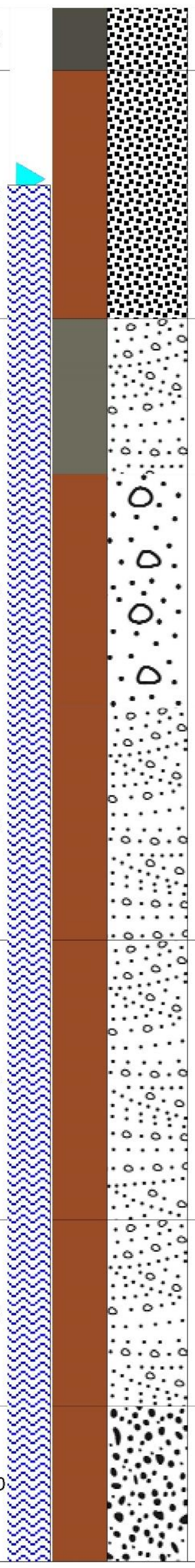
PROJECT: Geotechnical Investigation on Karuma-Lira T/Line

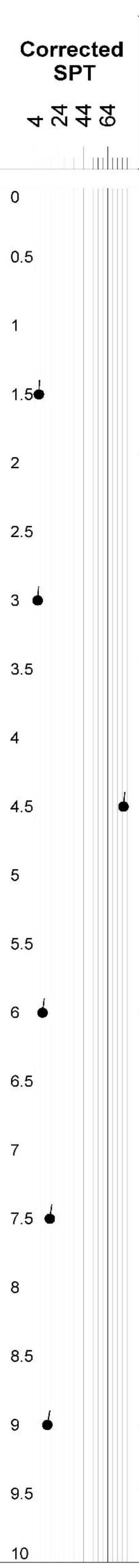
CLIENT: Samuel Acidri

GETLAB LTD — YOUR RELIABLE ENGINEERING LABORATORY

PROJECT NUMBER:	**P2017014**	**BH LOCATION: B103+05**		**START DATE:**	**N/A**
BH REF:	**G/BH/17/0456**			**COMPLETION DATE:**	**N/A**
OFFSET:	**0.2 m**	**DRILLED DEPTH (m): 10**		**EASTING:**	**426605.777**
DISTRICT:	**Oyam**			**NORTHING:**	**246178.091**
WATER TABLE (m):	**1.14 m**	**TEST METHOD: BRITISH STANDARDS**		**UTM Zone:**	**36 N**
				SURFACE:	**Grassland**

BOREHOLE LEGEND / SOIL DESCRIPTION / GEOTECHNICAL DESIGN PARAMETERS





Depth (m)	Soil description	N (blows)	N_{55} (blows)	ρ (g/cm³)	LL (%)	PI (%)	< 0.075 mm (%)	G_S	NMC (%)	Cl^- (%)	SO_4^{2-} (%)	pH	Group Classification
0.00-0.40	moist dark grey soft sandy CLAYS			1.681	38.5	15.2	76.4	2.462	24.8	0.014	0.954	5.22	CI
0.40-2.00	moist reddish brown mottled greyish hard sandy CLAYS			1.685	37.8	15.8	70	2.453	23.7	0.014	0.958	5.18	CI
		10	7	1.741	38.6	15.2	65.3	2.556	24.3	0.014	0.966	5.25	CI
2.00-3.00	moist greyish mottled yellowish brown hard gravelly CLAYS			1.745	35.9	19.6	46.9	2.593	16.5	0.007	0.854	5.16	CI
3.00-4.50	moist reddish brown mottled grey loose dense silty GRAVELS	6	6										
				2.241	18.7	NP	1.7	2.739	12.5	0.009	1.146	5.45	GM
4.50-6.00	moist reddish brown mottled grey hard gravelly CLAYS	100	77	1.763	38.4	21.8	48.8	2.612	18.6	0.009	1.245	5.63	CI
				1.765	39.6	20.9	45.3	2.614	19.6	0.012	1.212	5.65	CI
6.00-7.80	moist reddish brown mottled grey stiff gravelly CLAYS	12	10										
				1.785	37.3	19.4	51.2	2.597	21.2	0.013	1.463	5.89	CI
				1.754	36.5	19.9	49.7	2.591	20.1	0.012	1.452	5.91	CI
		18	16										
7.80-9.00	moist reddish brown mottled grey stiff gravelly CLAYS -with cobbles of weathered quarzite			1.763	38.2	21.2	54.3	2.597	15.7	0.014	1.698	6.06	CI
9.00-10.00	moist reddish brown mottled olive grey stiff sandy SILTS -with completely weathered rock	16	14	1.745	42.6	9.6	69.8	2.587	28.6	0.015	1.543	5.91	MI

NMC-Natural Moisture Content, **PI**-Plasticity Index, **LL**-Liquid Limit, **SO_4^{2-}**-Sulphates, **Cl^-**-Chlorides, **BH**-Borehole, **SPT**-Standard Penetration Test, **G_s**-Specific Gravity, **N**-measured SPT value, **N_{corr}**-Corrected SPT value, **NP**-Non Plastic **p**-Bulk Density

Table B.1-2: Boring Log summary for AP 108/15

BOREHOLE LOG	Borehole No: **AP 108/15**	Geotechnical Lab:	GEOTECH SOLUTIONS
Commencement Date:	N/A	Easting Coordinate:	**391575.254**
Completion Date:	N/A	Northing Coordinate:	**194777.676**
Drilling Equipment:	**GY-50**	Logged by:	**John Richard Odeke**
Driller:	**LA**	Drilling Method:	**Rotary Method**
Final Hole Depth (m):	**10.0**	Core Diameter (mm):	**100**
Hole Inclination:	**90**	Client:	**Samuel Acidri**

Depth (m)		**Run Length (m)**	**Core Recovery (m)**	**Core Recovery (%)**	**RQD (m)**	**RQD (%)**	**Water Table**	**Graphic Log**	**Description**	**SPT**			
										10 cm	**10 cm**	**10 cm**	**N**
0.00	1.00	1.00	1						Greyish black dense Clayey SAND				
	1.45	0.45							SPT Test No. 1	12	10	9	31
	2.45	1.00	1						Grey dense Clayey SAND				
	2.90	0.45							SPT Test No. 2	14	16	14	44
	3.90	1.00	1						Grey medium dense Silty SAND				
	4.20	0.30					Not Encountered		Shelby No. 1				
	4.65	0.45							SPT Test No. 3	7	8	9	24
5.00													
	5.65	1.00	1						Conglomeratic Sandstone				
	6.65	1.00	1										
	7.65	1.00	1										
	8.56	1.00	1										
	9.65	1.00	1										
10.00		0.35											
End of borehole													

LEGEND:

Silty SAND

Clayey SAND

Conglomeratic sandstone

Table B.1-3: Boring Log summary for AP 104/5

BOREHOLE LOG	Location No: **AP 104/5**	Geotechnical Lab:	GEOTECH SOLUTIONS
Commencement Date:	**9/4/2019**	Easting Coordinate:	**400765.294**
Completion Date:	**9/4/2019**	Northing Coordinate:	**212510.472**
Drilling Equipment:	**GY-50 / Dando Terrier**	Logged by:	**J.R. Odeke/Jerome O.**
Drill Superintendent:	**Eric M**	Drilling Method:	**Rotary Method**
Final Hole Depth (m):	**20.0**	Core Diameter (mm):	**120**
Hole Inclination:	**90**	Client:	**Samuel Acidri**

Ref Depth (m)	Elevation (m)	Depth (m)	General Description	Graphic Log	Run length	Core recovery (m)	Core recovery (%)	RQD (m)	RQD (%)	Water Table	Permeability (m/sec)	Q (l/min)	Lugeon (Lu)	SPT 100 mm	SPT 100 mm	SPT 100 mm	SPT N	REMARKS
0.00	1131.00	0.00	Blackish brown top soil							0.3m								
	1130.70	0.30	Moist brown soft inorganic Sandy Lean CLAY		1.0	1.0	100.0											
1.00	1130.00	1.00												1	1	1	3	**SPT 1**
			Moist greyish brown firm inorganic Sandy Lean CLAY		1.0	1.0	100.0											
2.00																		
					1.0	1.0	100.0											
	1128.70	2.30												1	3	3	7	**SPT 2**
			Moist greyish brown firm inorganic Sandy Lean CLAY															
3.00																		
					1.0	1.0	100.0											
	1127.40	3.60												2	4	5	11	**SPT 3**
			Wet grey medium dense Clayey SAND															
4.00					1.0	1.0	100.0											
4.90	1126.10	4.90												3	5	6	14	**SPT 4**

Ref Depth (m)	Elevation (m)	Depth (m)	General Description	Graphic Log	Run length	Core recovery (m)	Core recovery (%)	RQD (m)	RQD (%)	Water Table	Permeability (m/sec)	Q (l/min)	Lugeon (Lu)	SPT				REMARKS
5.00			Wet brownish grey medium dense Clayey SAND		1.0	1.0	100.0											
6.00					1.0	1.0	100.0											
	1124.80	6.20												5	6	7	18	SPT 5
			Wet brownish grey medium dense Clayey SAND															
7.00					1.0	1.0	100.0											
	1123.50	7.50												5	7	8	20	SPT 6
			Moist greyish brown very stiff inorganic Sandy Lean CLAY															
8.00					1.0	1.0	100.0											
	1122.20	8.80												5	8	8	21	SPT 7
9.00			Moist greyish brown hard inorganic Sandy Lean CLAY		1.0	1.0	100.0											
10.00																		
	1120.90	10.10												8	13	16	37	SPT 8
			Slightly moist greyish brown medium dense Clayey SAND		1.0	1.0	100.0											
11.00					1.0	1.0	100.0											
	1119.60	11.40												9	14	15	38	SPT 9
			Slightly moist greyish brown medium dense Clayey SAND															
11.90																		

Ref Depth (m)	Elevation (m)	Depth (m)	General Description	Graphic Log	Run length	Core recovery (m)	Core recovery (%)	RQD (m)	RQD (%)	Water Table	Permeability (m/sec)	Q (l/min)	Lugeon (Lu)	SPT				REMARKS
12.00	1118.30	12.70	Slightly moist greyish brown medium dense Clayey SAND		1.0	1.0	100.0							8	9	10	27	SPT 10
13.00 **14.00**	1117.00	14.00	Slightly moist greyish brown medium dense Clayey SAND		1.0	1.0	100.0							8	9	11	28	SPT 11
15.00	1115.70	15.30	Slightly moist greyish brown medium dense Clayey SAND		1.0	1.0	100.0							9	12	16	37	SPT 12
16.00	1114.40	16.60	Slightly moist greyish brown medium dense Clayey SAND		1.0	1.0	100.0							9	12	14	35	SPT 13
					1.0	1.0	100.0											
17.00 **18.00**	1113.10	17.90	Slightly moist brownish grey dense Silty SAND		1.0	1.0	100.0							11	13	15	39	SPT 14
18.90			Slightly moist brownish grey dense Silty SAND		1.0	1.0	100.0											

Ref Depth (m)	Elevation (m)	Depth (m)	General Description	Graphic Log	Run length	Core recovery (m)	Core recovery (%)	RQD (m)	RQD (%)	Water Table	Permeability (m/sec)	Q (l/min)	Lugeon (Lu)	SPT				REMARKS
19.00																		
	1111.80	19.20	Slightly moist brownish grey dense Silty SAND		1.0	1.0	100.0							11	15	17	43	SPT 15
20.00	1111.00	20.00																

End of borehole

Legend

Top Soil

Sandy Lean CLAY

Silty SAND

Clayey SAND

Ground water table was encountered at 0.3m

Appendix B.2 - Borehole Core Logs and Photos

Table B.2-1: Investigation Borehole Core Photos for KL 30 (B103+5)

Project: **Geotechnical Investigations for KL 30 (B103+5)- 132KV TL**		
Client:	Soil Investigation Laboratory	Location
Samuel & Sinohydro	**GETLAB (U) Ltd**	**KL 30 (B103+5)**

Sample boxes for KL 30 (B103+5) from 0.00-10.00 m

Standard penetration test samples for KL 30 (B103+5)

Table B.2-2: Investigation Borehole Core Photos for AP 108/15

Project: **Geotechnical Investigations for AP 108/15 - 400KV TL**			
Client:		Soil Investigation Laboratory	Location
Samuel Acidri	**Geotech Solutions (U) Ltd**		**AP 108/15**

Depth: 0.0m - 5.0m

Depth: 5.0m - 10.0m

Table B.2-3: Boring Log summary for AP 108/20

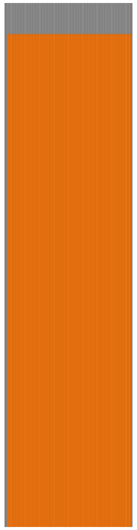
<table>
<tr><td rowspan="3">Geotechnical Investigations Boring Log</td><td colspan="2">Geotechnical Soil Laboratory:</td><td colspan="2">Location Coordinates</td></tr>
<tr><td>Profiled by: R. Sembera</td><td rowspan="2">Trial Pit No: AP108/20</td><td>Easting</td><td>Northing</td></tr>
<tr><td>Date: N/A</td><td>391873.5</td><td>192810.1</td></tr>
<tr><td>Pit profile</td><td>
</td><td>0.0 - 0.1m
3.0m</td><td></td><td>Gray stiff silty soil
Brownish-orange laterite, with duricrust.
Hard to excavate.
Ground water table not encountered</td></tr>
<tr><td>Excavation Method: Manual
Water Table: Not encountered</td><td>Geotech Solutions (U) Ltd
P.O. Box 4849 Kampala
Plot 42, Buikwe Road, Njeru Town</td><td>Location area terrain:
Flat to gently sloping</td><td colspan="2"></td></tr>
</table>

Table B.2-4: Investigation Borehole Core Photos for AP 104/5

Project: **Geotechnical Investigations for AP 104/5 - 400KV TL**		
Client:	Soil Investigation Laboratory	Location
Samuel Acidri	**Geotech Solutions (U) Ltd**	**AP 104/5**

Depth: 0.0m - 5.0m

Depth: 5.0m - 10.0m

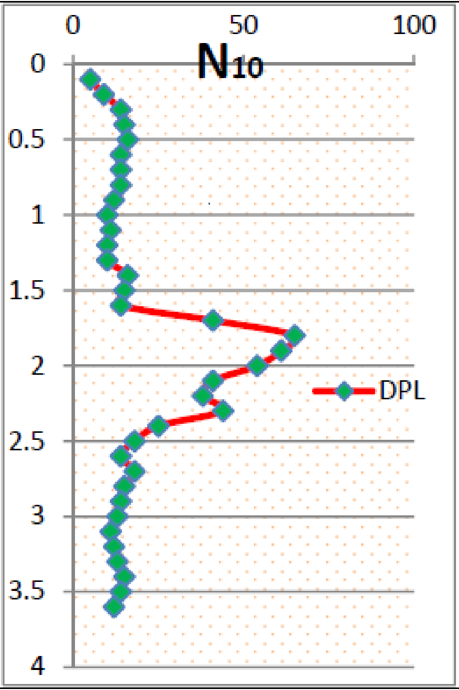
Appendix B.3 - DPL Results

Table B.3-1: Dynamic Probing Light Test (DPL) Summary for AP 108/20

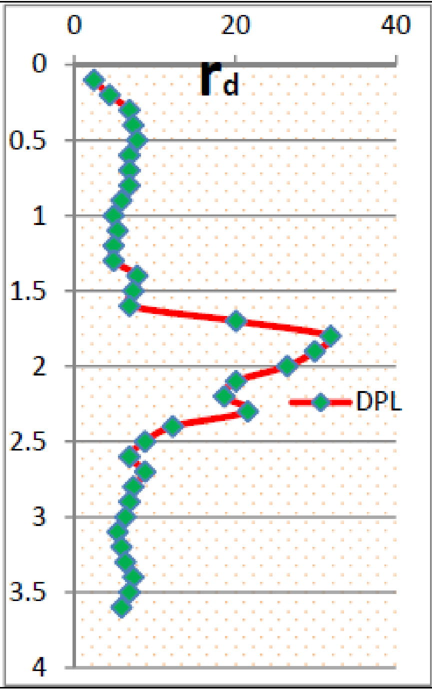
Client: Samuel Acidri		Dynamic Probing Light		GEOTECH SOLUTIONS
Project Area	**Easting**	**Northing**	**Location**	**Date**
Kawanda-Karuma TL	391873.474	192810.148	AP 108/20	N/A
Cone area = 0.001 m^2		Hammer = 10 kg	Fall = 500 mm	
Depth	Blows per 10cm Penetration	Penetration Rate	Unit Point Resistance	Dynamic Point Resistance
(m)	N_{10}	e (m per blow)	r_d (MPa)	q_d (MPa)
0				
0.1	5	0.020	2.45	1.4
0.2	9	0.011	4.41	2.5
0.3	14	0.007	6.86	3.8
0.4	15	0.007	7.35	4.1
0.5	16	0.006	7.84	4.4
0.6	14	0.007	6.86	3.8
0.7	14	0.007	6.86	3.8
0.8	14	0.007	6.86	3.8
0.9	12	0.008	5.88	3.3
1	10	0.010	4.9	2.7
1.1	11	0.009	5.39	2.7
1.2	10	0.010	4.9	2.5
1.3	10	0.010	4.9	2.5
1.4	16	0.006	7.84	3.9
1.5	15	0.007	7.35	3.7
1.6	14	0.007	6.86	3.4
1.7	41	0.002	20.09	10.0
1.8	65	0.002	31.85	15.9
1.9	61	0.002	29.89	14.9
2	54	0.002	26.46	13.2
2.1	41	0.002	20.09	9.1
2.2	38	0.003	18.62	8.5
2.3	44	0.002	21.56	9.8
2.4	25	0.004	12.25	5.6
2.5	18	0.006	8.82	4.0
2.6	14	0.007	6.86	3.1
2.7	18	0.006	8.82	4.0
2.8	15	0.007	7.35	3.3

2.9	14	0.007	6.86	3.1
3	13	0.008	6.37	2.9
3.1	11	0.009	5.39	2.2
3.2	12	0.008	5.88	2.5
3.3	13	0.008	6.37	2.7
3.4	15	0.007	7.35	3.1
3.5	14	0.007	6.86	2.9
3.6	12	0.008	5.88	2.5
Supervisor/in-charge: **John Richard Odeke**			Geologist: **R. Sembera**	

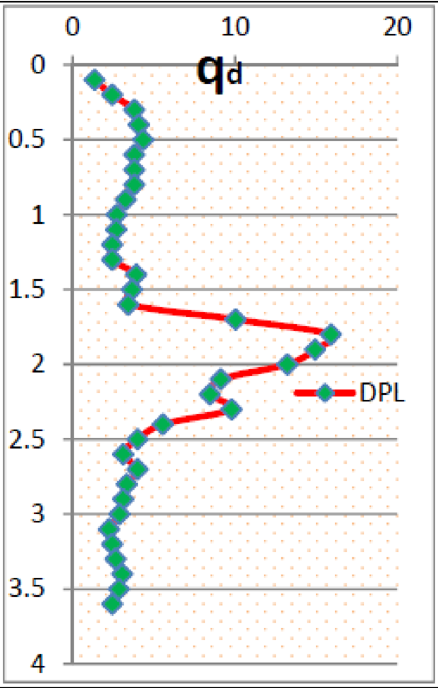
Depth Versus N_{10} Graph



Depth Versus r_d Graph



Depth Versus q_d Graph



Dynamic Penetrometer Light (DPL) Test [EN-ISO-22476-2:2002]

Appendix B.4 - SPT Results

Table B.4-1: Bearing capacity from SPT values for KL 30 (B103+5)

ALLOWABLE BEARING CAPACITY FROM SPT RESULTS FOR KL 30 (B103+5)																		
Borehole No	**SPT Depth (m)**	**No. of seating and driving blows**	**Measured SPT N-values**	**Soil Consistency**	**Overburden correction Factor (Burt, 2007)**	**Hammer factor**	**Rod Length factor**	**Sample factor**	**Borehole Diameter factor**	**Overall correction factor, $C_{ER} = C_N * \eta_1 * \eta_2 * \eta_3$**	**Corrected SPT N-Value**	**Allowable Bearing Capacity, q_{all} (kPa) — Foundation Width, B (m)**						
			N		**C_N**	**η_1**	**η_2**	**η_3**	**η_4**	**C_{ER}**	**N_{55}**	**1**	**2**	**3**	**4**	**5**	**6**	**7**
KL 30 (B103+5)	1.50 - 1.95	6, 7, 3	10	Firm	1.00	0.91	0.75	1.00	1.00	0.68	7	186	144	123	114	108	104	102
	3.00 - 3.45	3, 3, 3	6	Loose	1.38	0.91	0.75	1.00	1.00	0.94	6	160	132	121	108	101	96	93
	4.50 - 4.95	Refusal	100	Hard	1.00	0.91	0.85	1.00	1.00	0.77	77	2048	1693	1549	1479	1403	1324	1269
	6.00 - 6.45	5, 6, 6	12	Stiff	1.00	0.91	0.95	1.00	1.00	0.86	10	266	220	201	192	187	183	174
	7.50 - 7.95	4, 6, 12	18	Stiff	1.00	0.91	0.95	1.00	1.00	0.86	16	426	352	322	307	299	293	289
	9.00 - 9.45	5, 7, 9	16	Stiff	1.00	0.91	0.95	1.00	1.00	0.86	14	372	308	282	269	262	257	253

Table B.4-2: Bearing capacity from SPT corrected values for AP 108/15

BEARING CAPACITY RESULTS FROM SPT VALUES FOR AP 108/15									
Borehole No	**Depth (m)**	**Field SPT N-values**	**Rod Length correction (η_2)**	**Overburden Correction (C_N)**	**N_{55}**	**Soil Description**	**Consistency**	**q_{ult} (kPa)**	**q_{all} (kPa)**
AP 108/15	0-1.00	31.0	0.75	2.28	43	Clayey SAND	Dense	1457.1	485.7
	1.00-2.45	44.0	0.75	1.45	39	Clayey SAND	Dense	1321.3	440.4
	2.45-4.20	24.0	0.85	1.11	19	Silty SAND	Medium Dense	623.8	207.9
	4.20-5.65	No SPT due to presence of boulders							

Table B.4-3: Bearing Capacity Evaluation based on corrected field SPT N-values for AP 104/5

GEOTECH SOLUTIONS	BEARING CAPACITY BASED ON CORRECTED FIELD SPT N-VALUES FOR AP 104/5											
Project:	Geotechnical Investigation for AP 104/5 Karuma-Kawanda Transmission Line (400 kV)											
Client:	Samuel Acidri											
Location:	AP 104/5 (400 kV Karuma-Kawanda TL)											
Sampling Date:	N/A		**Testing Date:**		10/6/2019 to 15/6/2019			**Hammer Weight (kg):**		63.5		
Technician:	J.R. Odeke		**Checked by:**		BK			**Depth:**		1.00m – 19.20m		
BOWLES' APPROACH												
Location	**Depth (m)**	**Field SPT N-values**	**Rod Length correction** η_2	**Unit Weight (kN/m^2)**	**Effective Unit Weight**	p'_o **(kPa)**	**Overburden correction** C_N	N_{55}	**Soil Description**	**Consistency**	**Allowable Bearing Pressure q_a, (kPa)** B = 1.0m	**Allowable Bearing Pressure q_a, (kPa)** B = 2.0m
AP 104/5	1.00	3	0.75	18.00	8.19	11.13	1.00	2	Sandy Lean CLAY	Very Soft	53.2	38.5
	2.30	7	0.75	18.00	8.19	21.78	1.00	4	Sandy Lean CLAY	Soft	106.4	87.9
	3.60	11	0.75	18.00	8.19	32.43	1.00	7	Sandy Lean CLAY	Firm	186.2	153.9
	4.90	14	0.85	18.50	8.69	43.72	1.48	14	Clayey SAND	Stiff	372.4	307.8
	6.20	18	0.95	18.50	8.69	55.02	1.32	18	Clayey SAND	Very Stiff	478.8	395.8
	7.50	20	0.95	18.50	8.69	66.32	1.20	19	Clayey SAND	Very Stiff	505.4	417.7
	8.80	21	0.95	18.50	8.69	77.62	1.00	16	Sandy Lean CLAY	Very Stiff	425.6	351.8
	10.10	37	1.00	18.50	8.69	88.91	1.00	30	Sandy Lean CLAY	Hard	798.0	659.6

	11.40	38	1.00	18.50	8.69	100.21	0.98	30	Clayey SAND	Medium Dense	798.0	659.6
	12.70	27	1.00	18.50	8.69	111.51	0.93	20	Clayey SAND	Medium Dense	532.0	439.7
	14.00	28	1.00	18.50	8.69	122.80	0.88	20	Clayey SAND	Medium Dense	532.0	439.7
	15.30	37	1.00	18.50	8.69	134.10	0.85	26	Clayey SAND	Medium Dense	691.6	571.7
	16.60	35	1.00	18.50	8.69	145.40	0.81	23	Silty SAND	Medium Dense	611.8	505.7
	17.90	39	1.00	18.50	8.69	156.69	0.78	25	Silty SAND	Medium Dense	665.0	549.7
	19.20	43	1.00	18.50	8.69	167.99	0.76	27	Silty SAND	Medium Dense	718.2	593.6

Remarks:

1) $N'_{55} = C_N \text{ x } N \text{ x } \eta_1 \text{ x } \eta_2 \text{ x } \eta_3 \text{ x } \eta_4$
2) Allowable Bearing Pressure, $q_a = (N/F_1)K_d$ where $B < F_4$; $q_a = N/F_2\left((B+F_3)/B\right)^2 K_d$ where $B > F_4$; $K_d = 1 + \frac{0.33D}{B} \leq 1.33$; $F_1 = 0.05$; $F_2 = 0.08$; $F_3 = 0.3$ and $F_4 = 1.2$
3) For SPT indicated as R, a value of N = 100 was assumed.
4) These results relate to the points that were tested.

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Technical Manager

Appendix B.5 - Soil Resistivity Test Results

Table B.5-1: Soil Resistivity Measurement Summary for KL 30 (B103+5)

SOIL RESISTIVITY SURVEY RESULTS FOR KL 30 (B103+5) -(IEEE Std 80-2000/BS 1377: Part 9:1990/BS 5930: 1990)													
Tower No	**Average Temperature**	**a = 0.30m**		**a = 1.00m**		**a = 2.00m**		**a = 3.00m**		**a = 4.00m**		**a = 5.00m**	
	℃	**R (Ω)**	***pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**
KL 30 (B103+5)	39	123.3	232	45.2	284	15.4	193	11.2	211	9.5	238	5.2	165

Table B.5-2: Soil Resistivity Survey Results for AP 108/15

SOIL RESISTIVITY SURVEY RESULTS FOR AP 108/15												
Tower No	**a (m)**	**0.3**	**a (m)**	**1.0**	**a (m)**	**2.0**	**a (m)**	**3.0**	**a (m)**	**4.0**	**a (m)**	**5.0**
	R (Ω)	***pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**
AP108/15	0.27	0.51	1.30	8.17	0.61	7.65	2.30	43.35	0.25	6.29	3.60	113.10

Table B.5-3: Soil Resistivity Survey Results for AP 108/20

SOIL RESISTIVITY SURVEY RESULTS FOR AP 108/20												
Tower No	**a (m)**	**0.3**	**a (m)**	**1.0**	**a (m)**	**2.0**	**a (m)**	**3.0**	**a (m)**	**4.0**	**a (m)**	**5.0**
	R (Ω)	***pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**	**R (Ω)**	***Pa* (Ωm)**
AP 108/20	598.4	1127.96	276.3	1736.04	131.5	1652.48	70.8	1334.55	35.4	889.70	14.9	468.10

Where: a = Electrode spacing (m); R = measured resistance (Ω); and pa = apparent resistivity (Ωm).

Table B.5-4: Soil Resistivity Survey Result for AP 104/5

SOIL RESISTIVITY SURVEY RESULTS FOR AP 104/5											
Easting:		400765.294		**Foundaion Location:**		AP 104/5		**Northing:**		212510.472	
a = Electrode spacing (m)	**0.3**	a = Electrode spacing (m)	**1.0**	a = Electrode spacing (m)	**2.0**	a = Electrode spacing (m)	**3.0**	a = Electrode spacing (m)	**4.0**	a = Electrode spacing (m)	**5.0**
R = measured resistance (Ω)	*pa* = apparent resistivity (Ωm)	R = measured resistance (Ω)	*pa* = apparent resistivity (Ωm)	R = measured resistance (Ω)	*pa* = apparent resistivity (Ωm)	R = measured resistance (Ω)	*pa* = apparent resistivity (Ωm)	R = measured resistanc e (Ω)	*pa* = apparent resistivity (Ωm)	R = measured resistance (Ω)	*pa* = apparent resistivity (Ωm)
1.50	2.83	6.60	41.47	62.40	784.14	42.50	801.11	2.70	67.86	0.44	13.74

Appendix B.6 - Soil Classification Tests

Table B.6-1: Soil Classification Summary for KL 30 (B103+5)

SOIL CLASSIFICATION SUMMARY FOR KL 30 (B103+5)

(USING BS 1377: PART 2: 1990 AND BS 5930:1999+A2:2010)

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Tower Location	Depth (m)	Particle Size Distribution																Atterberg limits				BS 5930 Soil classification	Soil Chemical Tests			Specific Gravity	Bulk Density
		63.0	50.0	37.5	28.0	20.0	14.0	10.0	6.3	5.0	2.0	1.18	0.600	0.425	0.300	0.150	0.075	NMC	LL	PL	PI		Cl^-	SO_4^{2-}	pH		
		(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)		(%)	(%)		(G_s)	(g/cm^3)
KL 30 (B103+5)	0.00-0.40	100	100	100	100	100	100	100	100	100	99.5	99.1	98.5	96.3	91.3	85.3	76.4	24.8	38.5	23.3	15.2	CI	0.014	0.954	5.22	2.462	1.681
	0.40-1.50	100	100	100	100	100	100	100	100	99.7	99.6	99.4	98.0	95.6	86.0	73.2	70.0	23.7	37.8	22.0	15.8	CI	0.014	0.958	5.18	2.453	1.685
	1.50-2.00	100	100	100	100	100	100	100	100	99.8	98.3	97.4	96.4	93.3	87.4	70.2	65.3	24.3	38.6	23.3	15.2	CI	0.014	0.966	5.25	2.556	1.741
	2.00-3.00	100	100	100	100	100	95.0	86.9	77.9	75.0	70.0	69.1	67.8	65.4	62.4	49.8	46.9	16.5	35.9	16.3	19.6	CI	0.007	0.854	5.16	2.593	1.745
	3.00-4.50	56.8	45.1	43.2	35.7	35.7	35.5	35.3	35.3	35.2	34.8	34.6	29.9	22.3	14.7	3.2	1.7	12.5	18.7	NP	NP	GM	0.009	1.146	5.45	2.739	2.241
	4.50-5.10	100	100	100	100	100	95.4	90.0	80.9	75.9	67.0	65.0	63.3	62.3	58.9	51.3	48.8	18.6	38.4	16.6	21.8	CI	0.009	1.245	5.63	2.612	1.763
	5.10-6.00	100	100	100	100	100	92.5	91.3	84.6	77.7	69.5	67.6	62.2	61.2	57.6	52.3	45.3	19.6	39.6	18.7	20.9	CI	0.012	1.212	5.65	2.614	1.765
	6.00-7.00	100	100	100	100	91.3	85.3	83.6	80.6	76.6	74.3	73.6	66.5	61.2	58.9	54.5	51.2	21.2	37.3	17.8	19.4	CI	0.013	1.463	5.89	2.597	1.785
	7.00-7.80	100	100	100	100	94.6	89.6	84.6	81.2	84.4	75.6	70.3	68.6	62.4	60.1	58.6	49.7	20.1	36.5	16.5	19.9	CI	0.012	1.452	5.91	2.591	1.754
	7.80-9.00	100	100	100	100	89.6	85.3	80.3	76.9	75.9	73.1	71.8	70.3	69.2	65.8	55.6	54.3	15.7	38.2	17.0	21.2	CI	0.014	1.698	6.06	2.597	1.763
	9.00-10.00	100	100	100	100	100	100	100	100	100	99.6	99.3	98.6	97.0	93.2	73.4	69.8	28.6	42.6	33.0	9.6	MI	0.015	1.543	5.91	2.587	1.745

Where:

NMC = Natural Moisture Content

LL = Liquid Limit

PL = Plastic Limit

PI = Plasticity Index

Cl^- = Chloride ions

SO_4^{2-} = Sulphate ions

Table B.6-2: Soil Indicator Test Results for KL 30 (B103+5)

SOIL INDICATOR TEST RESULTS FOR KL 30 (B103+5) (USING BS 1377: PART 2: 1990 AND BS 5930:1999+A2:2010)							
Tower Location	**Depth (m)**	**Gravel-sized particles (2.00-63 mm)**	**Sand-sized particles (0.075-2.00 mm)**	**Silt and clay sized particles (Less than 0.075 mm)**	**Plasticity Chart Description**	**Soil Description**	**Group Symbol (BS 5930)**
KL 30 (B103+5)	0.0 - 0.40	0.5	23.2	76.4	CI	Very sandy CLAYS of intermediate plasticity	CI
	0.40 - 1.50	0.4	29.6	70	CI	Very sandy CLAYS of intermediate plasticity	CI
	1.50 - 2.00	1.8	32.9	65.3	CI	Very sandy CLAYS of intermediate plasticity	CI
	2.00 - 3.00	30	23	46.9	CI	Gravelly CLAYS of intermediate plasticity	CI
	3.00 - 4.50	65.2	33.1	1.7	ML	Slightly silty GRAVELS of low plasticity	GM
	4.50 - 5.10	33	18.2	48.8	CI	Gravelly CLAYS of intermediate plasticity	CI
	5.10 - 6.00	30.6	24.2	45.3	CI	Gravelly CLAYS of intermediate plasticity	CI
	6.00 - 7.00	25.7	23.1	51.2	CI	Gravelly CLAYS of intermediate plasticity	CI
	7.00 - 7.80	24.4	26	49.7	CI	Sandy CLAYS of intermediate plasticity	CI
	7.80 - 9.00	26.9	18.8	54.3	CI	Gravelly CLAYS of intermediate plasticity	CI
	9.00 - 10.00	0.5	29.7	69.8	MI	Very sandy SILTS of intermediate plasticity	MI

Table B.6-3: Soil Classification Summary for AP 108/15

SOIL CLASSIFICATION SUMMARY FOR AP 108/15																
Tower Location	**Depth (m)**	**Moisture Content**	**Percentage Passing**							**Grading Modulus**	**Specific Gravity**	**Atterberg limits**			**USCS Classification**	**Group Symbol**
			25 (mm)	**19 (mm)**	**9.5 (mm)**	**4.75 (mm)**	**2.00 (mm)**	**425 (µm)**	**75 (µm)**			**LL %**	**PL %**	**PI %**		
AP 108/15	0.00 - 1.00	8.5%	100	100	100	98	96	75	60	0.69	2.777	32.80	18.00	14.80	Clayey SAND	SC
	1.00 - 2.45	7.0%	100	100	100	96	94	72	55	0.79	2.380	29.60	16.70	12.90	Clayey SAND	SC
	2.45 - 4.20	6.2%	100	100	100	99	95	76	36	0.93	2.370	24.70	12.50	12.20	Silty SAND	SM

Table B.6-4: Soil Classification Summary for AP 108/20

SOIL CLASSIFICATION SUMMARY FOR AP 108/20															
Tower Location	**Depth (m)**	**Specific Gravity**	**Moisture Content**	**Percentage Passing**							**Atterberg limits**			**USCS Classification**	**Group Symbol**
				25 (mm)	**19 (mm)**	**9.5 (mm)**	**4.75 (mm)**	**2.00 (mm)**	**425 (µm)**	**75 (µm)**	**LL %**	**PL %**	**PI %**		
AP 108/20	0.0-1.0	2.483	8.9%	100	100	87	62	41	30	23	29.8	18.2	11.6	Silty SAND with Gravel	SM
	1.0-2.0	2.380	9.0%	100	100	99	85	68	53	45	45.6	20.0	25.6	Silty SAND with Gravel	SM
	2.0-3.0	2.412	9.1%	100	100	97	60	30	22	20	44.8	21.4	23.4	Clayey SAND with Gravel	SC

Table B.6-5: Detailed Soil Classification Results for AP 104/5

DETAILED SOIL CLASSIFICATION RESULTS FOR AP 104/5

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Tower Location	Depth (m)	Moisture Content	Percentage Passing (Particle Size Distribution)																	Grading Modulus	Specific Gravity	Atterberg limits			USCS Classification	Group Symbol
		(%)	63 mm	50 mm	37.5 mm	28 mm	20 mm	14 mm	10 mm	6.3 mm	5.0 mm	2.36 mm	2.0 mm	1.18 mm	600 µm	425 µm	300 µm	150 µm	75 µm			LL (%)	PL (%)	PI (%)		
AP 104/5	0.0 – 1.0	15.6	100	100	100	100	100	99	95	90	86	85	83	78	72	69	66	59	56	0.92	2.65	30.7	14.4	16.3	Sandy Lean CLAY	CL
	1.0 – 3.5	14.2	100	100	100	100	100	100	100	100	99	93	90	88	86	85	82	75	69	0.55	2.66	42.8	19.4	23.4	Sandy Lean CLAY	CL
	3.5 – 5.0	11.6	100	100	100	100	100	100	100	100	100	100	100	100	96	88	78	60	47	0.65	2.71	35.5	15.4	20.1	Clayey SAND	SC
	5.0 – 8.0	12.5	100	100	100	100	100	100	100	100	100	100	100	100	88	74	69	53	44	0.82	2.70	32.6	14.8	17.8	Clayey SAND	SC
	8.0 – 10.0	17.4	100	100	100	100	100	100	100	100	100	100	100	100	99	98	90	76	68	0.34	2.64	39.9	24.3	15.6	Sandy Lean CLAY	CL
	10.0 – 12.0	15.5	100	100	100	100	100	100	100	100	100	100	100	100	97	95	78	66	63	0.42	2.66	35.8	19.0	16.8	Sandy Lean CLAY	CL
	12.0 – 15.0	13.5	100	100	100	100	100	100	100	99	99	98	98	97	97	96	94	62	45	0.61	2.71	34.5	14.8	19.7	Clayey SAND	SC
	15.0 – 16.3	12.6	100	100	100	100	100	100	100	99	99	98	98	97	97	92	88	66	47	0.63	2.69	35.2	18.6	16.6	Clayey SAND	SC
	16.3 – 18.5	11.8	100	100	100	100	98	98	98	97	97	96	96	96	96	95	95	64	46	0.63	2.72	28.5	NP	NP	Silty SAND	SM
	18.5 – 20.0	12.2	100	100	100	100	99	99	98	97	97	96	95	95	94	94	92	53	36	0.75	2.73	27.6	NP	NP	Silty SAND	SM

Table B.6-6: Summarised Soil Classification Test for AP 104/5

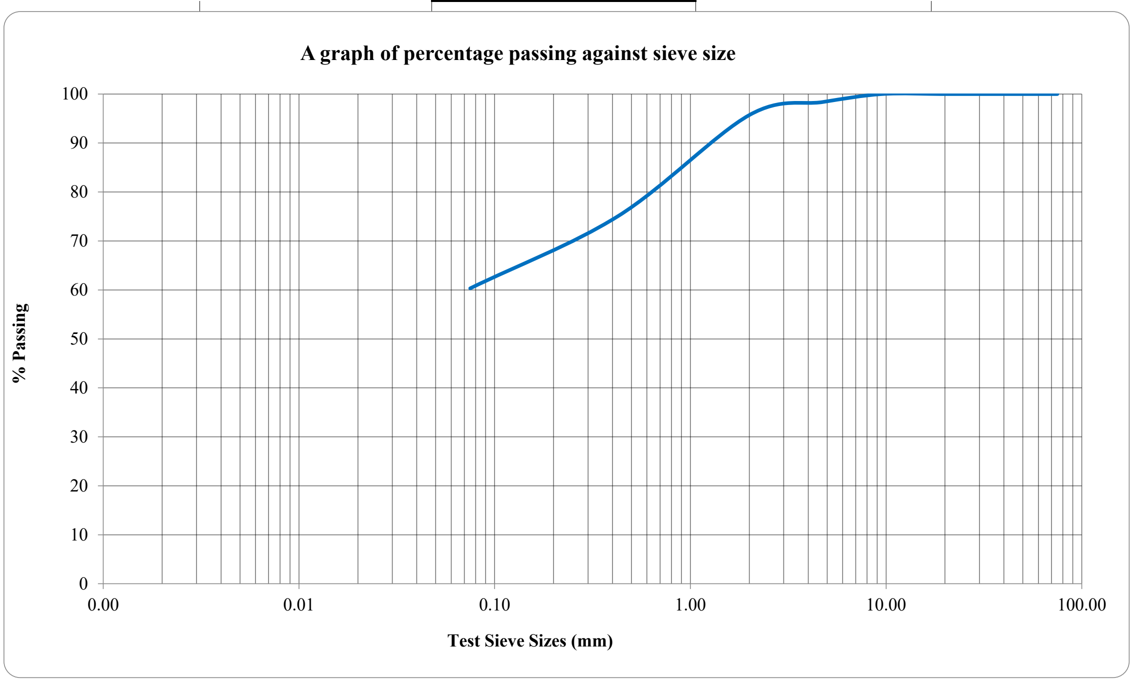
SUMMARISED SOIL CLASSIFICATION TEST FOR AP 104/5												
Tower Location	**Depth (m)**	**Moisture Content (%)**	**Gravel (%)**	**Sand (%)**	**Clay & Silt (%)**	**Grading Modulus**	**Atterberg limits**			**USCS Classification**	**Group Symbol**
							LL (%)	**PL (%)**	**PI (%)**		
AP 104/5	0.0 – 1.0	15.6	14	30	56	0.92	30.7	14.4	16.3	Sandy Lean CLAY	CL
	1.0 – 3.5	14.2	1	29	69	0.55	42.8	19.4	23.4	Sandy Lean CLAY	CL
	3.5 – 5.0	11.6	0	53	47	0.65	35.5	15.4	20.1	Clayey SAND	SC
	5.0 – 8.0	12.5	0	56	44	0.82	32.6	14.8	17.8	Clayey SAND	SC
	8.0 – 10.0	17.4	0	32	68	0.34	39.9	24.3	15.6	Sandy Lean CLAY	CL
	10.0 – 12.0	15.5	0	37	63	0.42	35.8	19.0	16.8	Sandy Lean CLAY	CL
	12.0 – 15.0	13.5	1	54	45	0.61	34.5	14.8	19.7	Clayey SAND	SC
	15.0 – 16.3	12.6	1	52	47	0.63	35.2	18.6	16.6	Clayey SAND	SC
	16.3 – 18.5	11.8	3	51	46	0.63	28.5	NP	NP	Silty SAND	SM
	18.5 – 20.0	12.2	3	61	36	0.75	27.6	NP	NP	Silty SAND	SM

PARTICLE SIZE DISTRIBUTION

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	0.0-1.0
Location:	Kigumba	**Testing Date:**	
Sampling Date:		**Test Method:**	ASTM D422-63
Pit No.:	AP108/15	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			

Initial wt before washing (g):	529.1	Moisture Content:	1.0
Dry wt after washing (g):	208.5	Initial Dry Weight (g):	524.0

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
25.0	0.0	0.0	0.0	100
19.0	0.0	0.0	0.0	100
9.50	0.0	0.0	0.0	100
4.75	8.5	8.5	1.6	98
2.00	14.0	22.5	4.3	96
0.425	108.4	130.9	25.0	75
0.075	77.0	207.9	39.7	60
pan	0.3	208.2		0.69



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
1.6	38.1	60.3

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

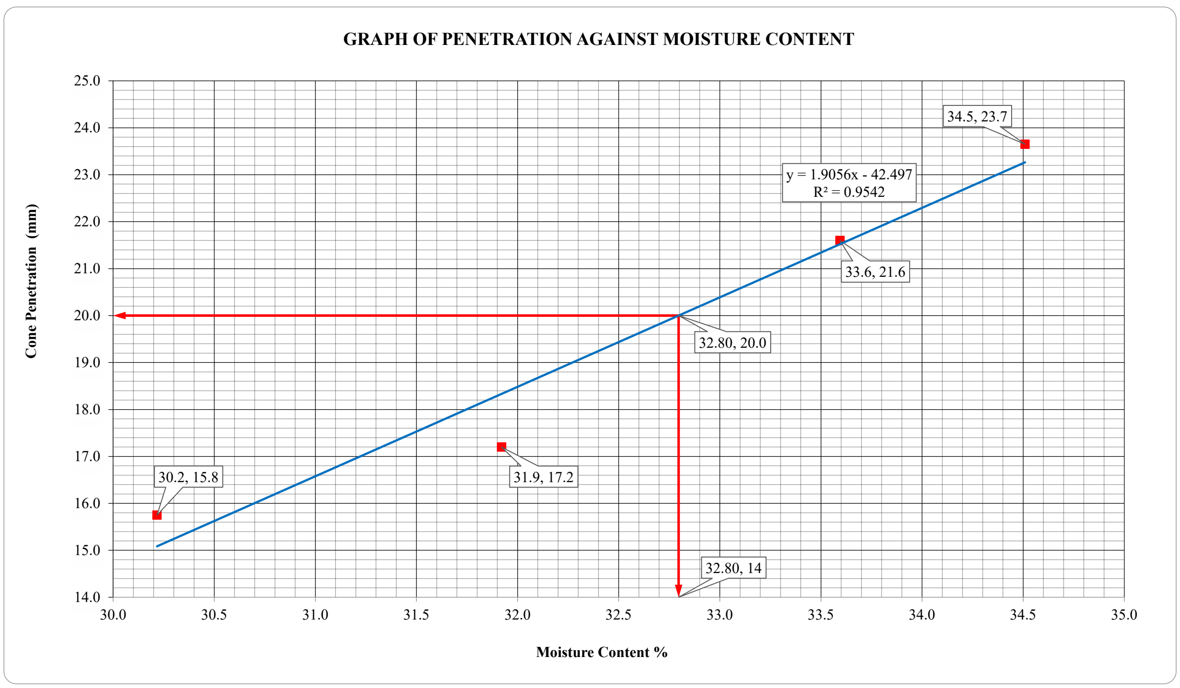
ATTERBERG LIMITS TEST REPORT

GEOTECH SOLUTIONS

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400 KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kigumba	**Depth (m):**	0.0-1.0
Sampling Date:		**Testing date:**	
Pit No.:	AP 108/15	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	Emma

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		**PLASTIC LIMIT (%)**	
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	15.7	15.8	17.1	17.3	21.5	21.7	23.6	23.7		
Average Penetration	mm	15.8		17.2		21.6		23.7			
Container Number		Z10	NB	S9	BT	S8	X7	TA	Z9	P5	S1
Weight of wet soil + container:	w2 (g)	42.2	39.2	43.5	43.7	45.7	47.2	40.7	41.8	32.1	33.4
Weight of dry soil + container:	w3 (g)	36.3	34.1	37.1	37.3	38.7	40.7	34.6	35.2	29.7	31.0
Weight of container:	w1 (g)	16.8	17.2	17.0	17.3	17.7	21.5	16.9	16.1	15.9	18.1
Weight of moisture:	w3 - w1 (g)	5.9	5.1	6.4	6.4	7.0	6.5	6.1	6.6	2.4	2.4
Weight of dry soil:	w2 - w3 (g)	19.5	16.9	20.1	20.0	21.0	19.2	17.7	19.1	13.8	12.9
Moisture content: {(w2-w3) / (w3-w1)} *100		30.3	30.2	31.8	32.0	33.3	33.9	34.5	34.6	17.4	18.6
Average Moisture Content:		30.2		31.9		33.6		34.5		18.0	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	131.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	6.4
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	32.8	**Plasticity Index (%)**	14.8
Plastic Limit (%)	18.0		

Remarks: These results relate to the sample that was tested

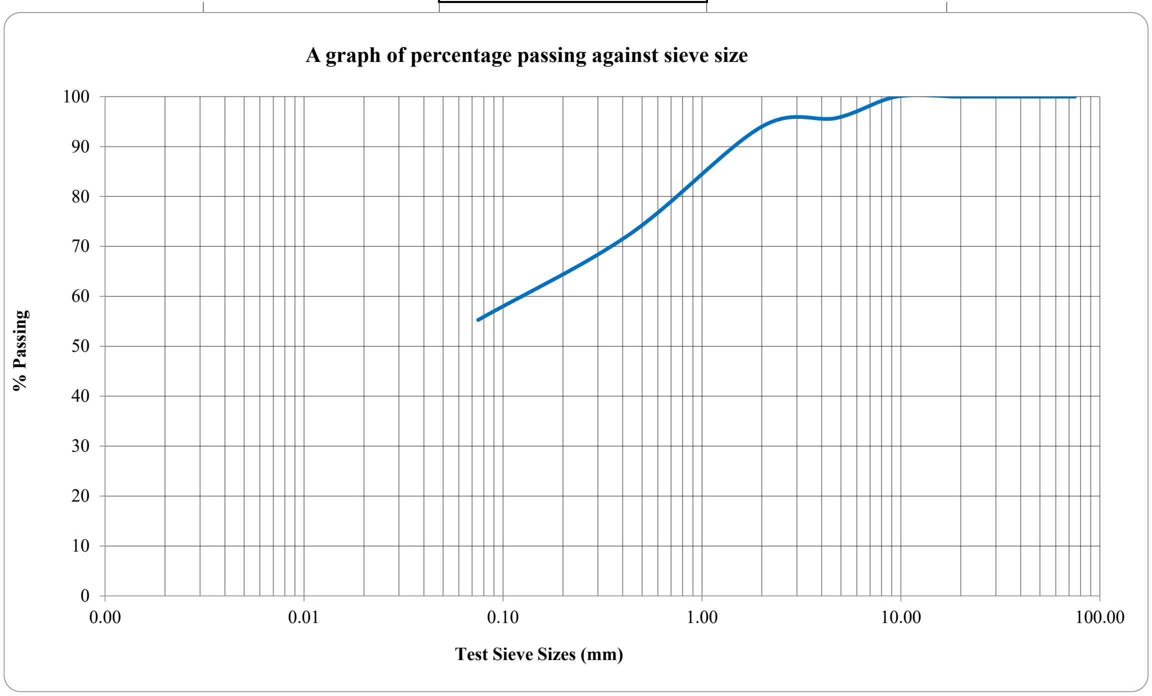
GEOTECH SOLUTIONS (U) LTD

Technical Manager

PARTICLE SIZE DISTRIBUTION

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	1.0-2.45
Location:	Kigumba	**Testing Date:**	
Sampling Date:		**Test Method:**	ASTM D422-63
Pit No.:	AP108/15	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			
Initial wt before washing (g):	300.1	Moisture Content:	3.5
Dry wt after washing (g):	130.2	Initial Dry Weight (g):	290.0

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
25.0	0.0	0.0	0.0	100
19.0	0.0	0.0	0.0	100
9.50	0.0	0.0	0.0	100
4.75	12.4	12.4	4.3	96
2.00	5.0	17.4	6.0	94
0.425	63.2	80.6	27.8	72
0.075	49.1	129.7	44.7	55
pan	0.3	130.0		0.79



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
4.3	40.5	55.3

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

ATTERBERG LIMITS TEST REPORT

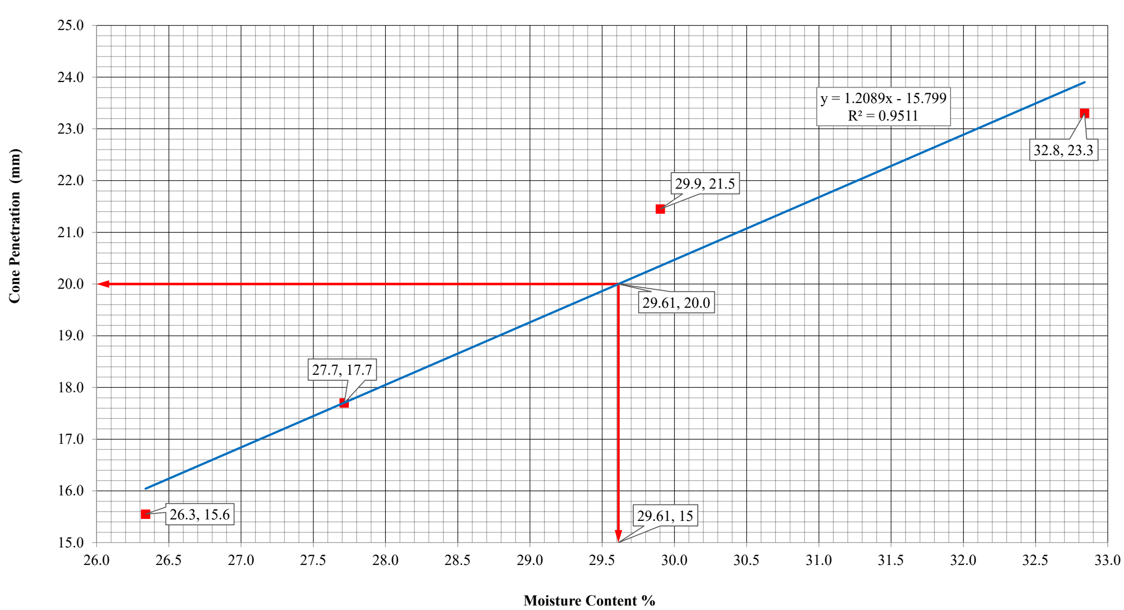
GEOTECH SOLUTIONS

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400 KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kigumba	**Depth (m):**	1.0-2.45
Sampling Date:		**Testing date:**	
Pit No.:	AP 108/15	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	Emma

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		PLASTIC LIMIT (%)	
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	15.5	15.6	17.6	17.8	21.4	21.5	23.2	23.4		
Average Penetration	mm	15.6		17.7		21.5		23.3			
Container Number		F	Q7	P5	U6	A	M3	Z1	D1	K10	K3
Weight of wet soil + container:	w2 (g)	54.4	57.6	55.9	51.6	61.8	55.2	61.4	56.3	25.6	28.3
Weight of dry soil + container:	w3 (g)	49.3	51.9	50.5	45.8	53.4	48.0	53.7	48.5	23.6	25.9
Weight of container:	w1 (g)	29.9	30.3	30.7	25.2	24.6	24.5	30.3	24.7	11.5	11.7
Weight of moisture:	w3 - w1 (g)	5.1	5.7	5.4	5.8	8.4	7.2	7.7	7.8	2.0	2.4
Weight of dry soil:	w2 - w3 (g)	19.4	21.6	19.8	20.6	28.8	23.5	23.4	23.8	12.1	14.2
Moisture content:	{(w2-w3) / (w3-w1)} *100	26.3	26.4	27.3	28.2	29.2	30.6	32.9	32.8	16.5	16.9
Average Moisture Content:		26.3		27.7		29.9		32.8		16.7	

GRAPH OF PENETRATION AGAINST MOISTURE CONTENT



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	128.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	8.6
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	29.6	**Plasticity Index (%)**	12.9
Plastic Limit (%)	16.7		

Remarks: These results relate to the sample that was tested

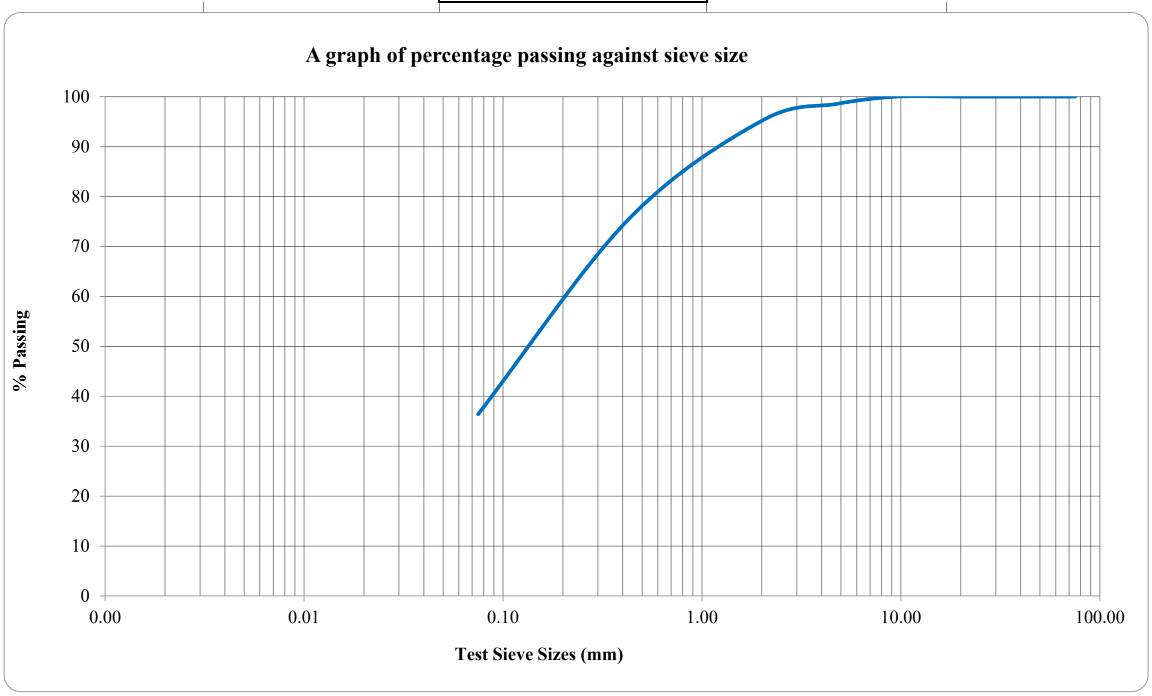
GEOTECH SOLUTIONS (U) LTD

Technical Manager

PARTICLE SIZE DISTRIBUTION

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	2.45-4.20
Location:	Kigumba	**Testing Date:**	
Sampling Date:		**Test Method:**	ASTM D422-63
Pit No.:	AP108/15	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			
Initial wt before washing (g):	589.2	Moisture Content:	0.9
Dry wt after washing (g):	372.2	Initial Dry Weight (g):	583.9

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
25.0	0.0	0.0	0.0	100
19.0	0.0	0.0	0.0	100
9.50	0.0	0.0	0.0	100
4.75	8.5	8.5	1.5	99
2.00	19.7	28.2	4.8	95
0.425	115.9	144.1	24.7	75
0.075	227.4	371.5	63.6	36
pan	0.3	371.8		0.93



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
1.5	62.2	36.4

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

GEOTECH SOLUTIONS

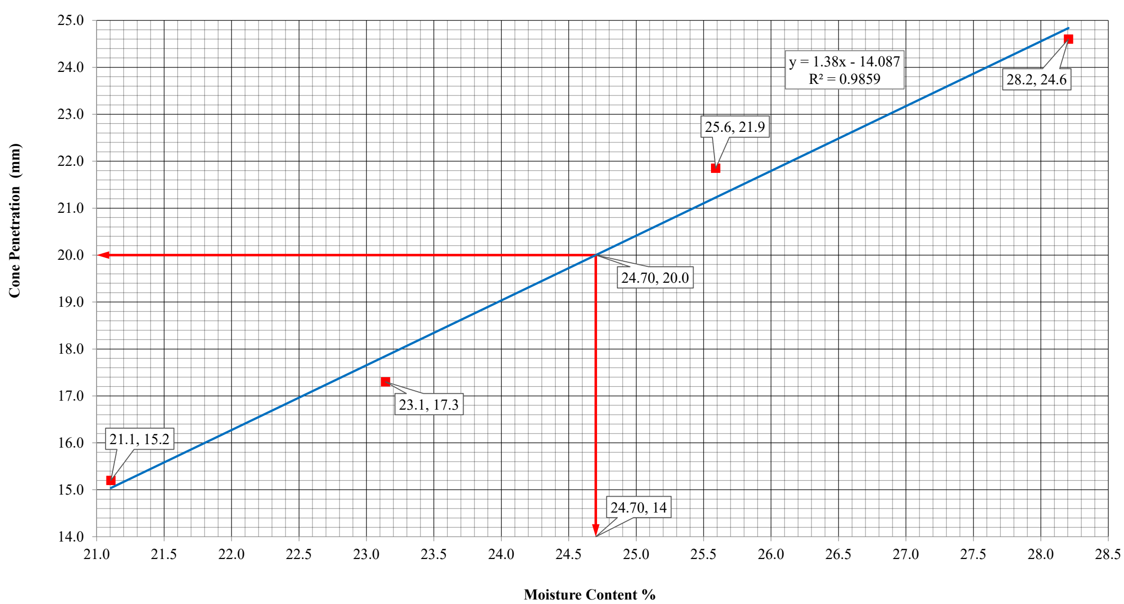
ATTERBERG LIMITS TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400 KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kigumba	**Depth (m):**	2.45-4.2
Sampling Date:		**Testing date:**	
Pit No.:	AP 108/15	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	Emma

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		PLASTIC LIMIT (%)	
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	15.1	15.3	17.2	17.4	21.8	21.9	24.5	24.7		
Average Penetration	**mm**	15.2		17.3		21.9		24.6			
Container Number		**E1**	**V5**	**I4**	**G3**	**N6**	**Y3**	**D7**	**S4**	**K12**	**K18**
Weight of wet soil + container:	**w2 (g)**	56.6	54.8	49.7	53.4	53.2	59.2	52.1	58.4	28.6	28.0
Weight of dry soil + container:	**w3 (g)**	52.1	50.5	45.0	49.1	47.5	53.1	46.1	52.3	26.7	26.2
Weight of container:	**w1 (g)**	30.7	30.2	24.6	30.6	24.9	29.6	24.8	30.7	11.1	12.1
Weight of moisture:	**w3 - w1 (g)**	4.5	4.3	4.7	4.3	5.7	6.1	6.0	6.1	1.9	1.8
Weight of dry soil:	**w2 - w3 (g)**	21.4	20.3	20.4	18.5	22.6	23.5	21.3	21.6	15.6	14.1
Moisture content: **{(w2-w3) / (w3-w1)} *100**		21.0	21.2	23.0	23.2	25.2	26.0	28.2	28.2	12.2	12.8
Average Moisture Content:		21.1		23.1		25.6		28.2		12.5	

GRAPH OF PENETRATION AGAINST MOISTURE CONTENT



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	133.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	5.0
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	24.7	**Plasticity Index (%)**	12.2
Plastic Limit (%)	12.5		

Remarks: These results relate to the sample that was tested

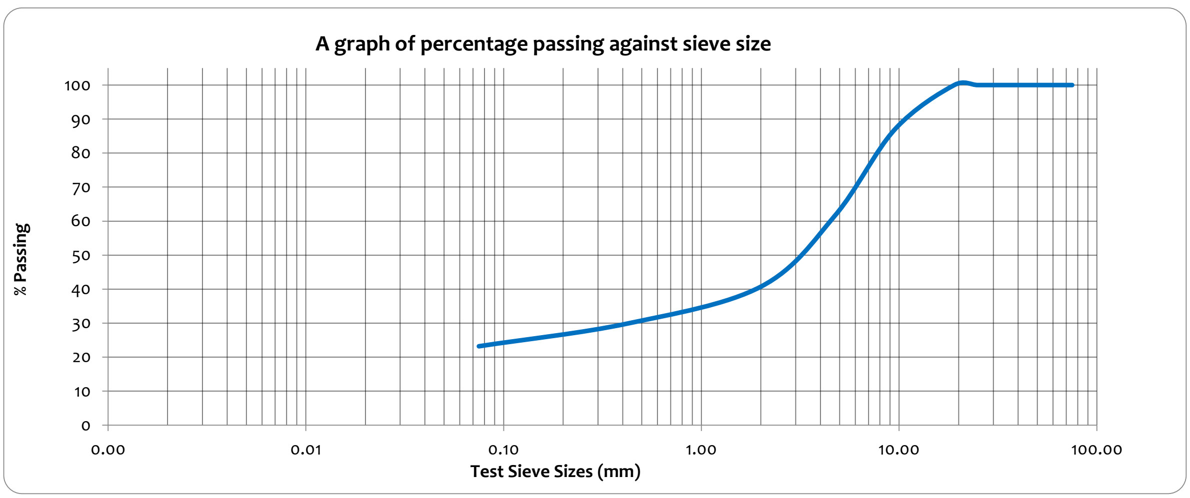
GEOTECH SOLUTIONS (U) LTD

Technical Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	400 KV KARUMA-KAWANDA POWER TRANSMISSION PROJECT		
Client:	SAMUEL ACIDRI	**Depth (m)**	0.00 - 1.00
Location:		**Test Method:**	ASTM D422 - 63
Pit No.:	AP108/20	**Technician:**	EMMA
Soil Description:	Clayey **SAND** with Gravel		
Initial wt before washing (g):	1381.8	Moisture Content:	1.5
Dry wt after washing (g):	1047.4	Initial Dry Weight (g):	1361.4

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.000	0.0	0.0	0.0	100
50.000	0.0	0.0	0.0	100
37.500	0.0	0.0	0.0	100
25.000	0.0	0.0	0.0	100
19.000	0.0	0.0	0.0	100
9.500	178.0	178.0	13.1	87
4.750	343.5	521.5	38.3	62
2.000	285.5	807.0	59.3	41
0.425	147.3	954.3	70.1	30
0.075	91.3	1045.6	76.8	23
pan	1.4	1047.0		2.1



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
38.3	38.5	23.2

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

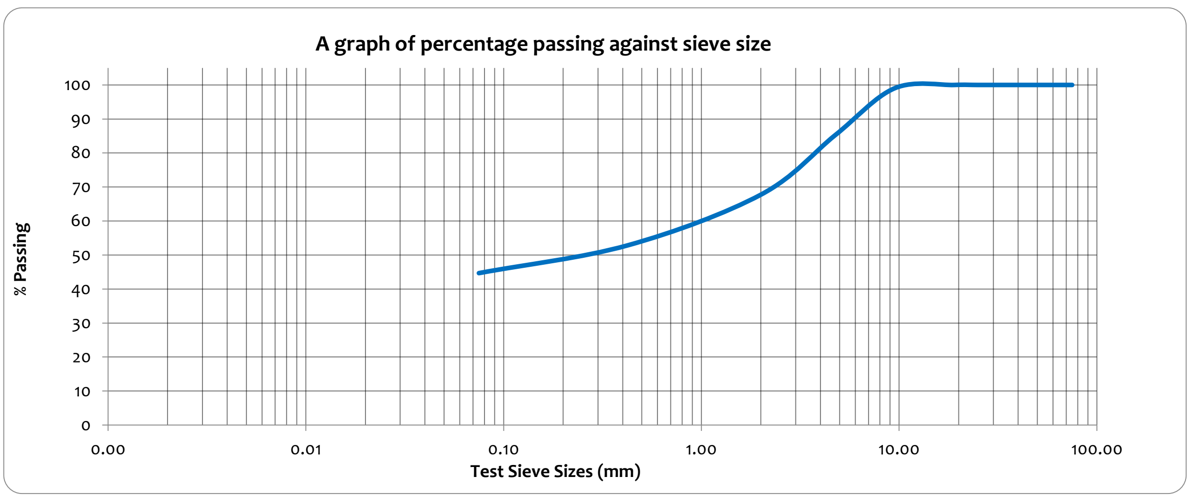
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	400 KV KARUMA-KAWANDA POWER TRANSMISSION PROJECT		
Client:	SAMUEL ACIDRI	**Depth (m)**	1.00 - 2.00
Location:		**Test Method:**	ASTM D422 - 63
Pit No.:	AP108/20	**Technician:**	EMMA
Soil Description:	Clayey **SAND** with Gravel		

Initial wt before washing (g):	1636.1	Moisture Content:	3.5
Dry wt after washing (g):	877.7	Initial Dry Weight (g):	1580.8

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.000	0.0	0.0	0.0	100
50.000	0.0	0.0	0.0	100
37.500	0.0	0.0	0.0	100
25.000	0.0	0.0	0.0	100
19.000	0.0	0.0	0.0	100
9.500	15.2	15.2	1.0	99
4.750	218.1	233.3	14.8	85
2.000	276.6	509.9	32.3	68
0.425	235.3	745.2	47.1	53
0.075	129.4	874.6	55.3	45
pan	0.7	875.3		1.3



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
14.8	40.6	44.7

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

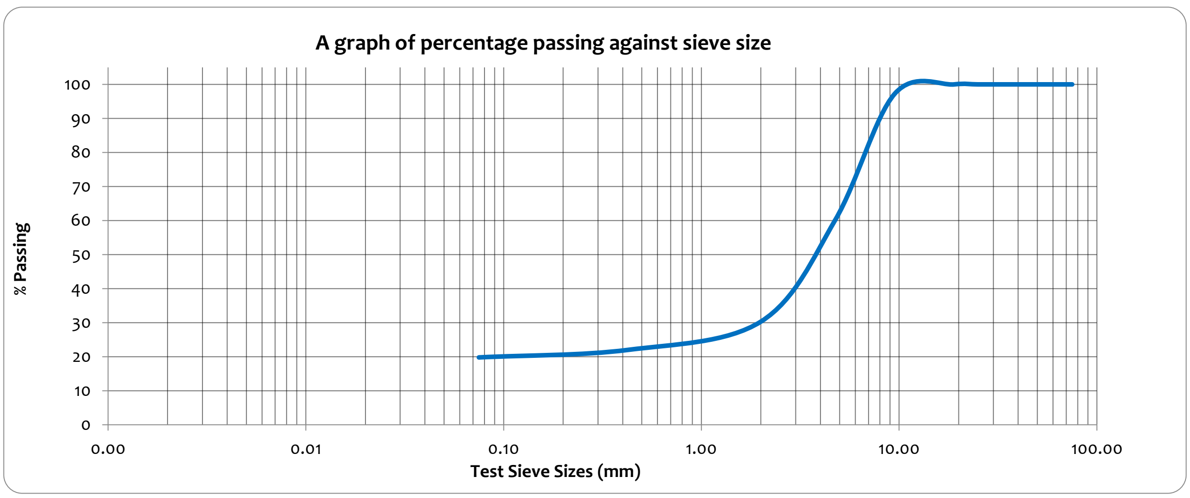
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	400 KV KARUMA-KAWANDA POWER TRANSMISSION PROJECT		
Client:	SAMUEL ACIDRI	**Depth (m)**	2.00 - 3.00
Location:		**Test Method:**	ASTM D422 - 63
Pit No.:	AP108/20	**Technician:**	EMMA
Soil Description:	Clayey **SAND** with Gravel		

Initial wt before washing (g):	1488.0	Moisture Content:	1.2
Dry wt after washing (g):	1182.3	Initial Dry Weight (g):	1470.4

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.000	0.0	0.0	0.0	100
50.000	0.0	0.0	0.0	100
37.500	0.0	0.0	0.0	100
25.000	0.0	0.0	0.0	100
19.000	0.0	0.0	0.0	100
9.500	40.5	40.5	2.8	97
4.750	545.7	586.2	39.9	60
2.000	438.6	1024.8	69.7	30
0.425	122.2	1147.0	78.0	22
0.075	32.5	1179.5	80.2	20
pan	1.3	1180.8		2.3



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
39.9	40.4	19.8

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

Atterberg Limits Data Sheet

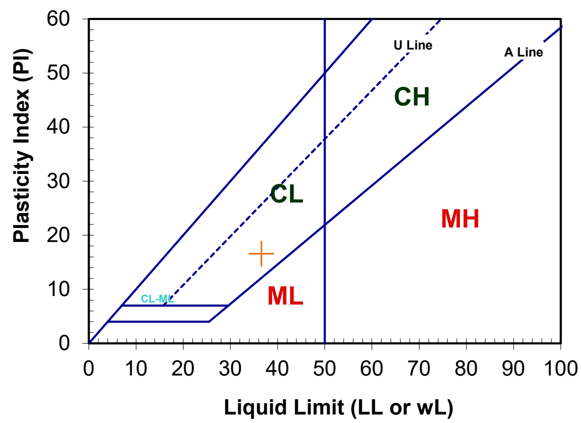
ASTM D4318-10

Project Name:	400 kV KARUMA-KAWANDA TL	**Tested By:**	EMMA	**Date:**
Location:	AP108/20	**Checked By:**		**Date:**
Client Name:	SAMUEL ACIDRI	**Test Number:**		
Sample Depth:	0.0 - 1.0 m	**Gnd Elevation:**		

USCS Soil Classification: ____________________

TEST			PLASTIC LIMIT				LIQUID LIMIT			
Variable	NO		1	2	3	4	1	2	3	4
	Var.	Units								
Number of Blows	N	blows					49	36	28	16
Can Number	---	---	P2-16.2				K19-11	P8-18.1	X8-19.4	P1-26.3
Mass of Empty Can	M_C	(g)	16.20				11.00	18.10	19.40	26.30
Mass Can & Soil (Wet)	M_{CMS}	(g)	17.40				22.00	29.30	30.00	41.00
Mass Can & Soil (Dry)	M_{CDS}	(g)	17.20				19.20	26.40	27.20	36.90
Mass of Soil	M_S	(g)	1.00				8.20	8.30	7.80	10.60
Mass of Water	M_W	(g)	0.20				2.80	2.90	2.80	4.10
Water Content	w	(%)	20.0				34.1	34.9	35.9	38.7

Liquid Limit (LL or w_L) (%):	36.60
Plastic Limit (PL or w_P) (%):	20.00
Plasticity Index (PI) (%):	16.60
USCS Classification:	CL



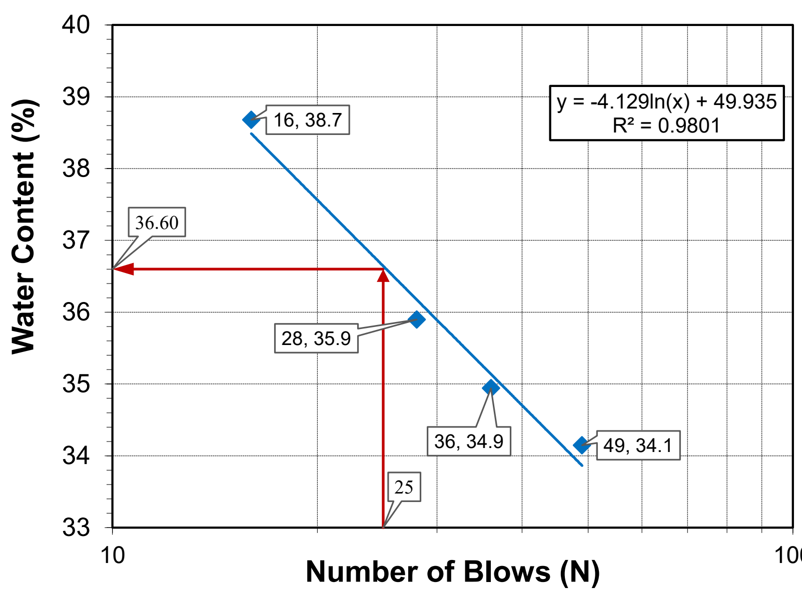
PI at "A" Line = 0.73(LL-20)

One Point Liquid Limit Calculation:

$LL = w_n(N/25)^{0.12}$

PROCEDURE USED

- [ ] Wet Preparation Multipoint
- [X] Dry Preparation Multipoint
- [ ] Procedure A Multipoint
- [ ] Procedure B One-Point



Atterberg Limits Data Sheet

ASTM D4318-10

Project Name:	400 kV KARUMA-KAWANDA TL	**Tested By:**	EMMA	**Date:**
Location:	AP108/20	**Checked By:**		**Date:**
Client Name:	SAMUEL ACIDRI	**Test Number:**		
Sample Depth:	1.0 - 2.0 m	**Gnd Elevation:**		

USCS Soil Classification:

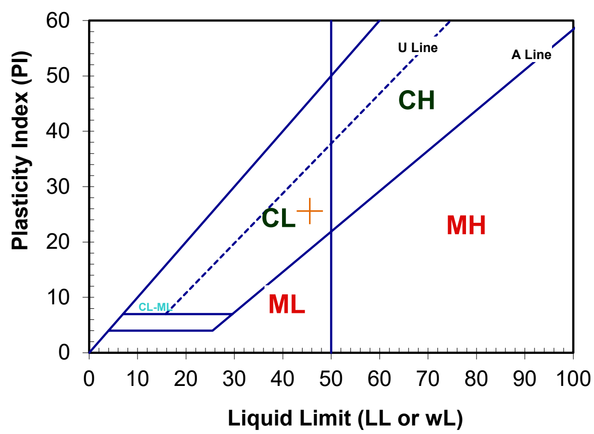
TEST			PLASTIC LIMIT				LIQUID LIMIT			
Variable	NO Var.	Units	1	2	3	4	1	2	3	4
Number of Blows	N	blows					48	38	28	17
Can Number	---	---	P2-16.2				W6-17.5	S6-15.8	A1-11	X2-17.4
Mass of Empty Can	M_C	(g)	16.20				17.50	15.80	11.00	17.40
Mass Can & Soil (Wet)	M_{CMS}	(g)	17.40				36.90	25.00	22.50	39.40
Mass Can & Soil (Dry)	M_{CDS}	(g)	17.20				31.40	22.30	18.90	32.20
Mass of Soil	M_S	(g)	1.00				13.90	6.50	7.90	14.80
Mass of Water	M_W	(g)	0.20				5.50	2.70	3.60	7.20
Water Content	w	(%)	20.0				39.6	41.5	45.6	48.6

Liquid Limit (LL or w_L) (%):	45.60
Plastic Limit (PL or w_P) (%):	20.00
Plasticity Index (PI) (%):	25.60
USCS Classification:	CL

PI at "A" Line = 0.73(LL-20)

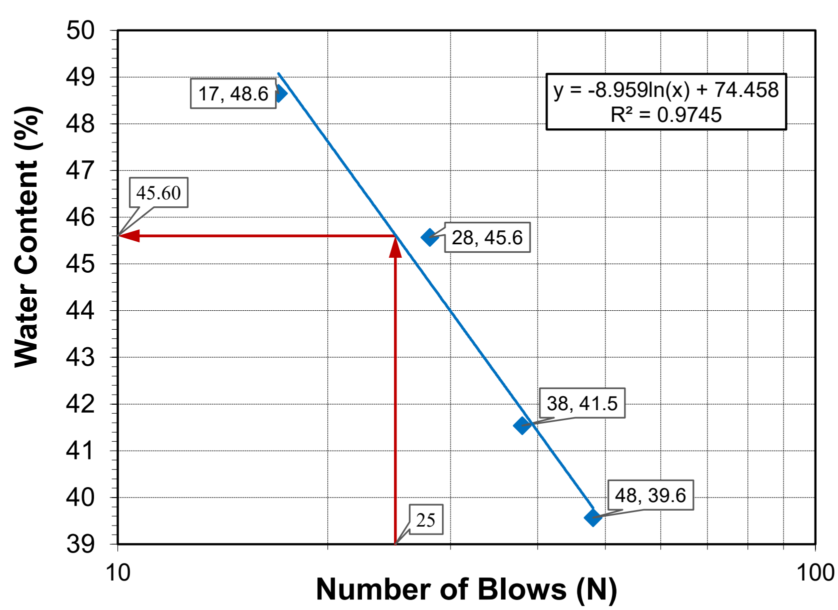
One Point Liquid Limit Calculation:

$LL = w_n(N/25)^{0.12}$



PROCEDURE USED

	Wet Preparation Multipoint
X	**Dry Preparation Multipoint**
	Procedure A Multipoint
	Procedure B One-Point



Atterberg Limits Data Sheet

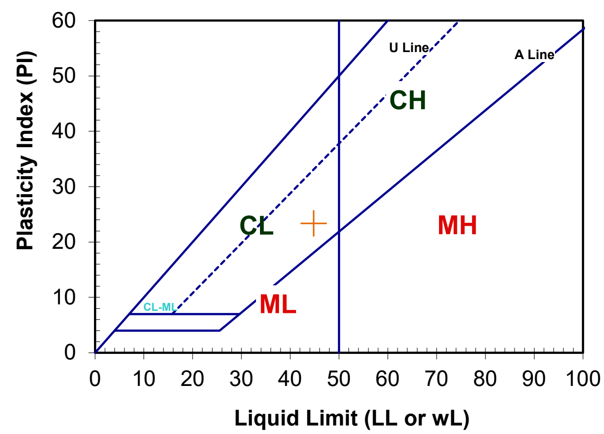
ASTM D4318-10

Project Name:	400 kV KARUMA-KAWANDA TL	**Tested By:**	EMMA	**Date:**	
Location:	AP108/20	**Checked By:**		**Date:**	
Client Name:	SAMUEL ACIDRI	**Test Number:**			
Sample Depth:	2.0 - 3.0 m	**Gnd Elevation:**			

USCS Soil Classification: ______________________

TEST			**PLASTIC LIMIT**				**LIQUID LIMIT**			
Variable	**NO**		**1**	**2**	**3**	**4**	**1**	**2**	**3**	**4**
	Var.	**Units**								
Number of Blows	N	blows					48	36	25	18
Can Number	---	---	Z8-17.6				S8-17.7	Z7-15.9	W7-15.6	Z10-16.5
Mass of Empty Can	M_C	(g)	17.60				17.70	15.90	15.60	16.50
Mass Can & Soil (Wet)	M_{CMS}	(g)	19.30				32.10	24.70	25.90	32.10
Mass Can & Soil (Dry)	M_{CDS}	(g)	19.00				28.00	22.10	22.80	27.00
Mass of Soil	M_S	(g)	1.40				10.30	6.20	7.20	10.50
Mass of Water	M_W	(g)	0.30				4.10	2.60	3.10	5.10
Water Content	w	(%)	21.4				39.8	41.9	43.1	48.6

Liquid Limit (LL or w_L) (%):	**44.80**
Plastic Limit (PL or w_P) (%):	**21.43**
Plasticity Index (PI) (%):	**23.37**
USCS Classification:	**CL**



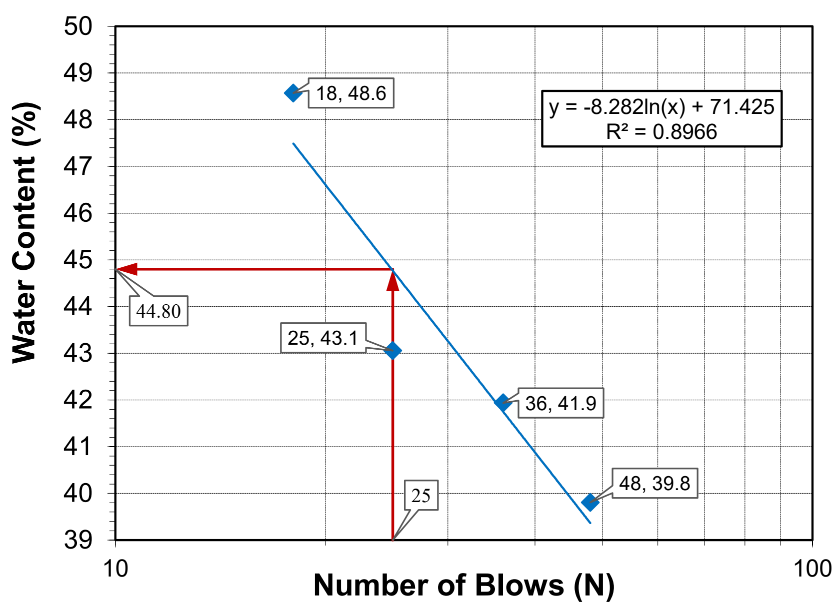
PI at "A" Line = 0.73(LL-20)

One Point Liquid Limit Calculation:

$LL = w_n (N/25)^{0.12}$

PROCEDURE USED

	Wet Preparation Multipoint
X	**Dry Preparation Multipoint**
	Procedure A Multipoint
	Procedure B One-Point

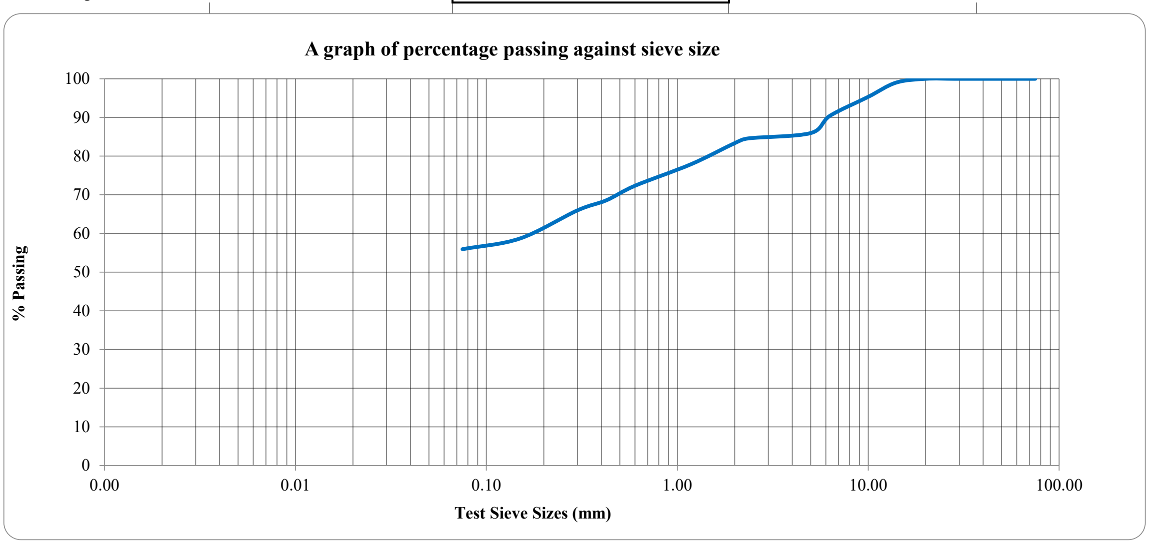


PARTICLE SIZE DISTRIBUTION TEST REPORT

GEOTECH SOLUTIONS

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	0.0-1.0
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			
Initial wt before washing (g):	550.5	Moisture Content:	0.9
Dry wt after washing (g):	242.1	Initial Dry Weight (g):	545.8

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	5.4	5.4	1.0	99
10.0	20.1	25.5	4.7	95
6.3	26.8	52.3	9.6	90
5.0	24.3	76.6	14.0	86
2.36	7.5	84.1	15.4	85
2.0	6.7	90.8	16.6	83
1.18	29.8	120.6	22.1	78
0.60	30.4	151.0	27.7	72
0.425	20.5	171.5	31.4	69
0.300	14.2	185.7	34.0	66
0.150	39.7	225.4	41.3	59
0.075	15.1	240.5	44.1	56
pan	1.2	241.7		0.92



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
14.0	30.0	55.9

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

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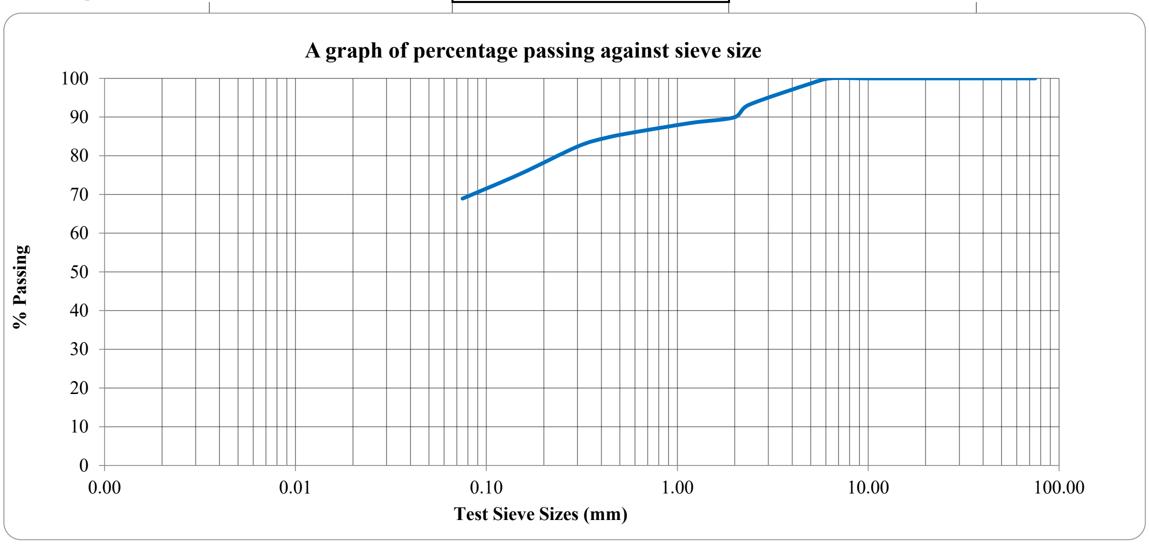
PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	1.0-3.5
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			

Initial wt before washing (g):	243.2	Moisture Content:	0.3
Dry wt after washing (g):	76.0	Initial Dry Weight (g):	242.4

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	0.0	0.0	0.0	100
10.0	0.0	0.0	0.0	100
6.3	0.0	0.0	0.0	100
5.0	3.2	3.2	1.3	99
2.36	13.5	16.7	6.9	93
2.0	7.6	24.3	10.0	90
1.18	3.6	27.9	11.5	88
0.60	5.8	33.7	13.9	86
0.425	3.5	37.2	15.3	85
0.300	5.5	42.7	17.6	82
0.150	17.2	59.9	24.7	75
0.075	15.4	75.3	31.1	69
pan	0.3	75.6		0.56



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
1.3	29.7	68.9

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

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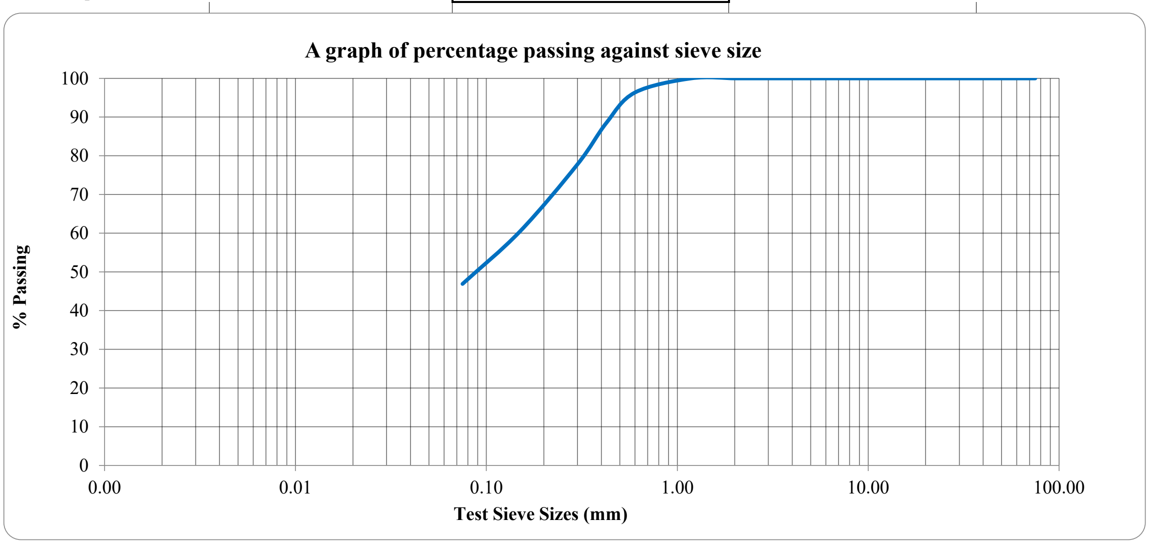
PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	3.5-5.0
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			
Initial wt before washing (g):	483.2	Moisture Content:	0.4
Dry wt after washing (g):	262.5	Initial Dry Weight (g):	481.4

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	0.0	0.0	0.0	100
10.0	0.0	0.0	0.0	100
6.3	0.0	0.0	0.0	100
5.0	0.0	0.0	0.0	100
2.36	0.0	0.0	0.0	100
2.0	0.0	0.0	0.0	100
1.18	0.0	0.0	0.0	100
0.60	17.6	17.6	3.7	96
0.425	37.8	55.4	11.5	88
0.300	51.5	106.9	22.2	78
0.150	83.4	190.3	39.5	60
0.075	65.3	255.6	53.1	47
pan	6.5	262.1		0.65



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
0.0	53.1	46.9

Remarks: These results relate to the sample that was tested

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Laboratory Manager

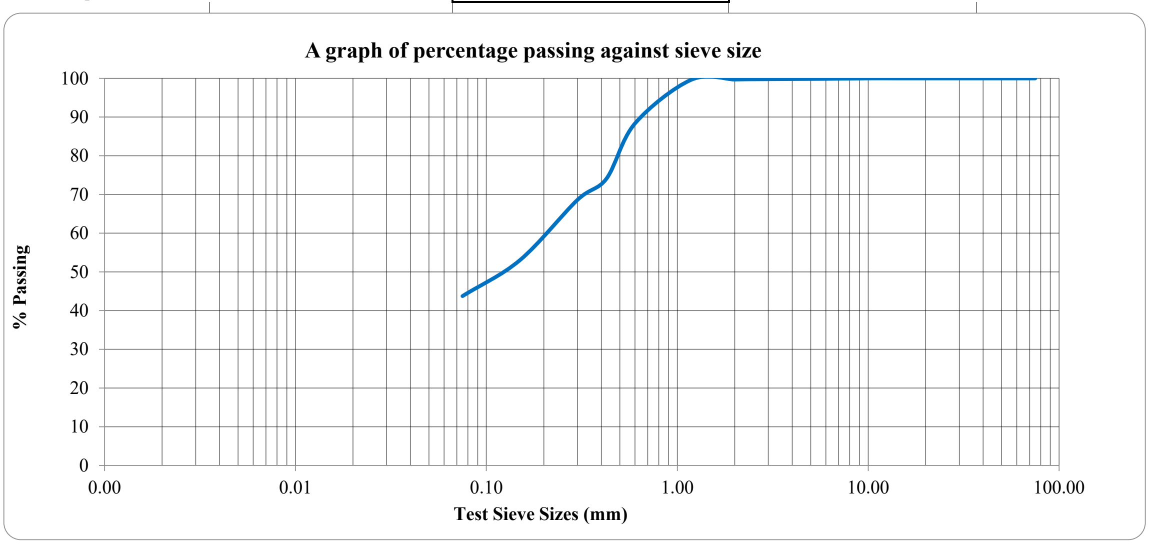
PARTICLE SIZE DISTRIBUTION TEST REPORT

GEOTECH SOLUTIONS

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	5.0-8.0
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			

Initial wt before washing (g):	550.4	Moisture Content:	0.4
Dry wt after washing (g):	316.0	Initial Dry Weight (g):	548.3

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	0.0	0.0	0.0	100
10.0	0.0	0.0	0.0	100
6.3	0.6	0.6	0.1	100
5.0	0.2	0.8	0.1	100
2.36	0.5	1.3	0.2	100
2.0	0.3	1.6	0.3	100
1.18	0.5	2.1	0.4	100
0.60	61.8	63.9	11.7	88
0.425	78.4	142.3	26.0	74
0.300	29.8	172.1	31.4	69
0.150	85.3	257.4	46.9	53
0.075	51.0	308.4	56.2	44
pan	7.4	315.8		0.82



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
0.1	56.1	43.8

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

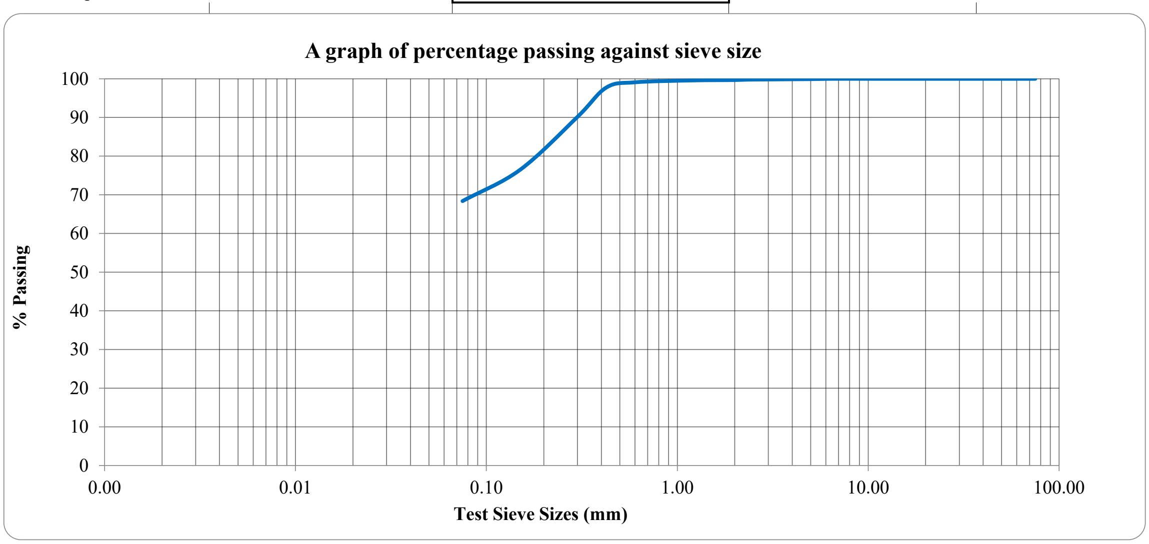
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

GEOTECH SOLUTIONS

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	8.0-10.0
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			

Initial wt before washing (g):		377.7	Moisture Content:	0.3
Dry wt after washing (g):		124.0	Initial Dry Weight (g):	376.7
Sieve Sizes (mm)	**Partial Retained Mass (g)**	**Cumulative Retained Mass (g)**	**Cumulative Retained (%)**	**% Passing**
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	0.0	0.0	0.0	100
10.0	0.0	0.0	0.0	100
6.3	0.0	0.0	0.0	100
5.0	0.3	0.3	0.1	100
2.36	0.5	0.8	0.2	100
2.0	0.4	1.2	0.3	100
1.18	0.4	1.6	0.4	100
0.60	1.8	3.4	0.9	99
0.425	5.1	8.5	2.3	98
0.300	28.6	37.1	9.8	90
0.150	51.6	88.7	23.5	76
0.075	30.4	119.1	31.6	68
pan	4.1	123.2		0.34



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
0.1	31.5	68.4

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

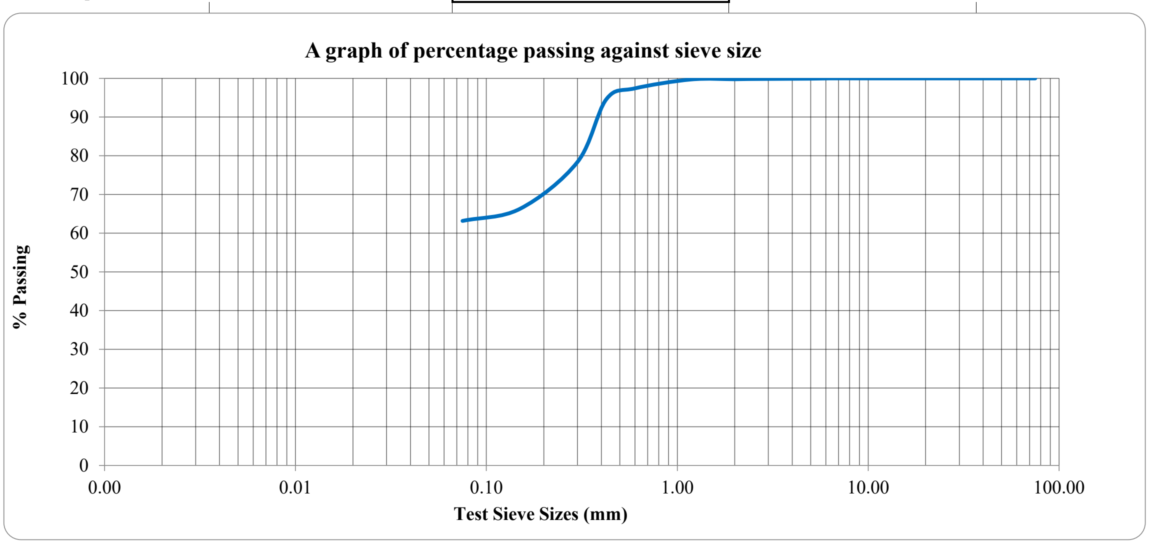
Laboratory Manager

GEOTECH SOLUTIONS

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	10.0-12.0
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			
Initial wt before washing (g):	545.2	Moisture Content:	0.3
Dry wt after washing (g):	204.9	Initial Dry Weight (g):	543.4

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	0.0	0.0	0.0	100
10.0	0.0	0.0	0.0	100
6.3	0.0	0.0	0.0	100
5.0	0.4	0.4	0.1	100
2.36	0.5	0.9	0.2	100
2.0	0.3	1.2	0.2	100
1.18	0.3	1.5	0.3	100
0.60	12.5	14.0	2.6	97
0.425	15.3	29.3	5.4	95
0.300	88.1	117.4	21.6	78
0.150	65.7	183.1	33.7	66
0.075	17.0	200.1	36.8	63
pan	4.3	204.4		0.42



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
0.1	36.8	63.2

Remarks: These results relate to the sample that was tested

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Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

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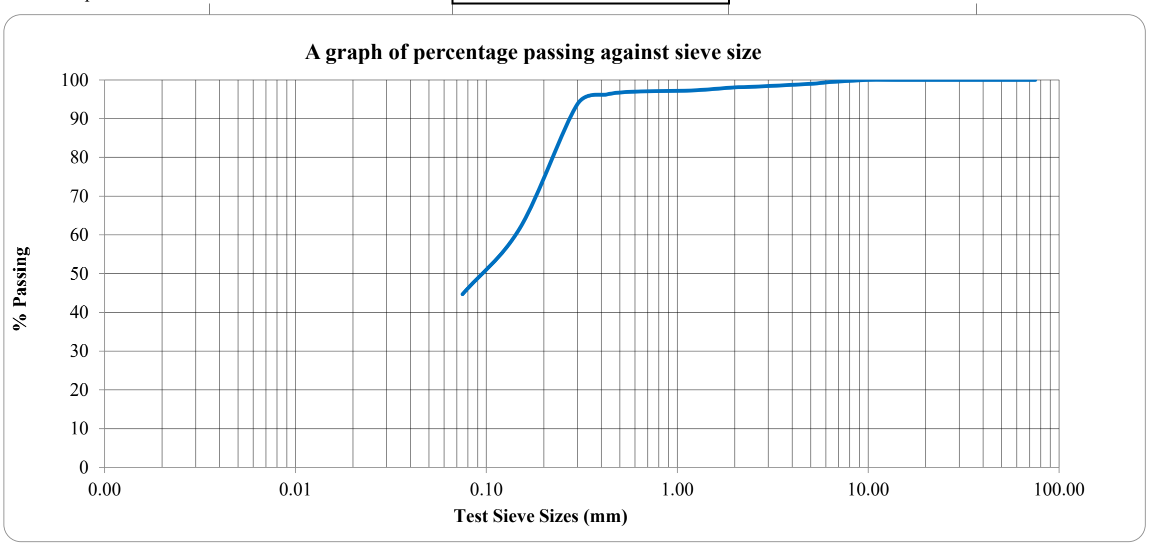
Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	12.0-15.0
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			

Initial wt before washing (g):	309.9	Moisture Content:	0.6
Dry wt after washing (g):	172.0	Initial Dry Weight (g):	308.1

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	0.0	0.0	0.0	100
10.0	0.0	0.0	0.0	100
6.3	1.7	1.7	0.6	99
5.0	1.4	3.1	1.0	99
2.36	2.5	5.6	1.8	98
2.0	0.3	5.9	1.9	98
1.18	2.5	8.4	2.7	97
0.60	0.9	9.3	3.0	97
0.425	2.3	11.6	3.8	96
0.300	7.6	19.2	6.2	94
0.150	98.5	117.7	38.2	62
0.075	52.7	170.4	55.3	45
pan	1.0	171.4		0.61



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
1.0	54.3	44.7

Remarks: These results relate to the sample that was tested

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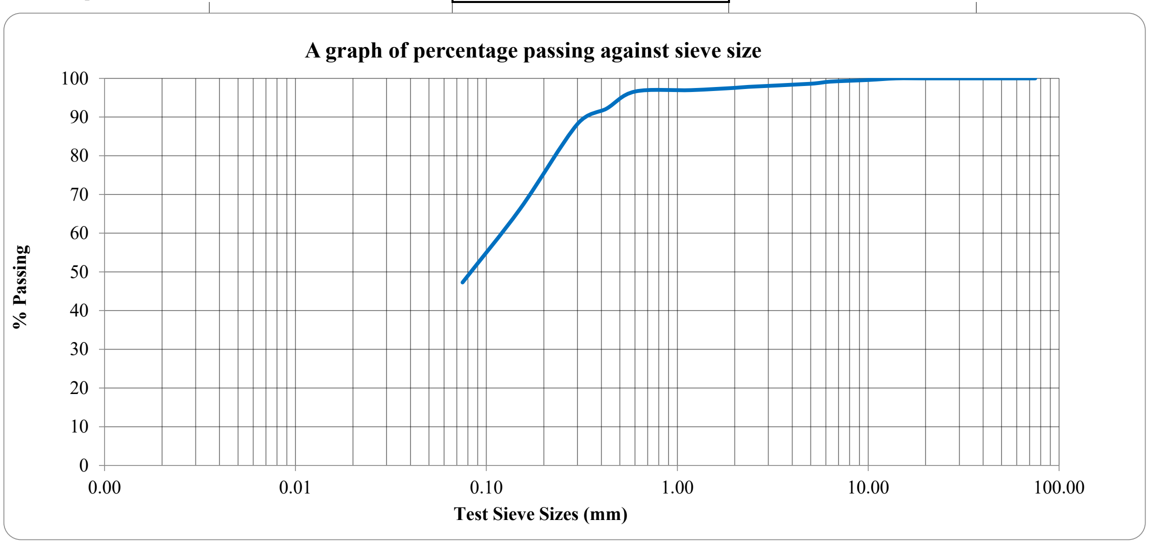
PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	15.0-16.3
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			

Initial wt before washing (g):	412.6	Moisture Content:	0.3
Dry wt after washing (g):	218.2	Initial Dry Weight (g):	411.6

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	0.0	0.0	0.0	100
14.0	0.0	0.0	0.0	100
10.0	1.7	1.7	0.4	100
6.3	1.8	3.5	0.9	99
5.0	2.2	5.7	1.4	99
2.36	3.3	9.0	2.2	98
2.0	1.1	10.1	2.5	98
1.18	2.4	12.5	3.0	97
0.60	1.5	14.0	3.4	97
0.425	18.3	32.3	7.8	92
0.300	16.2	48.5	11.8	88
0.150	90.4	138.9	33.7	66
0.075	78.1	217.0	52.7	47
pan	0.5	217.5		0.63



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
1.4	51.3	47.3

Remarks: These results relate to the sample that was tested

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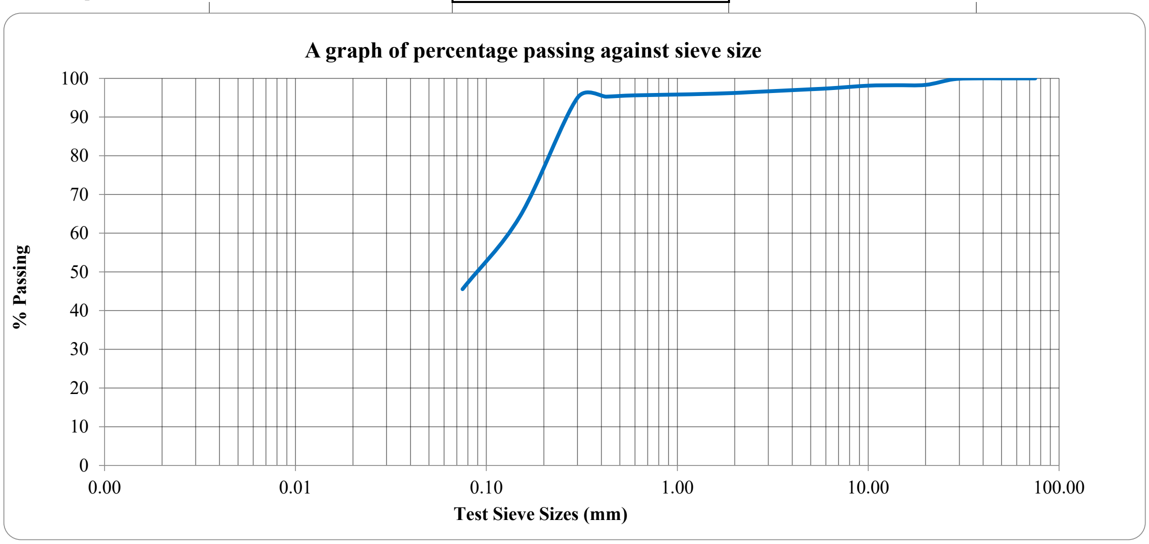
PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	16.3-18.5
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			

Initial wt before washing (g):	446.6	Moisture Content:	1.4
Dry wt after washing (g):	240.8	Initial Dry Weight (g):	440.3

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	1.0	1.0	0.2	100
20.0	6.4	7.4	1.7	98
14.0	0.4	7.8	1.8	98
10.0	0.5	8.3	1.9	98
6.3	3.0	11.3	2.6	97
5.0	1.1	12.4	2.8	97
2.36	3.4	15.8	3.6	96
2.0	0.8	16.6	3.8	96
1.18	1.5	18.1	4.1	96
0.60	1.2	19.3	4.4	96
0.425	1.5	20.8	4.7	95
0.300	1.3	22.1	5.0	95
0.150	134.8	156.9	35.6	64
0.075	82.9	239.8	54.5	46
pan	0.7	240.5		0.63



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
2.8	51.6	45.5

Remarks: These results relate to the sample that was tested

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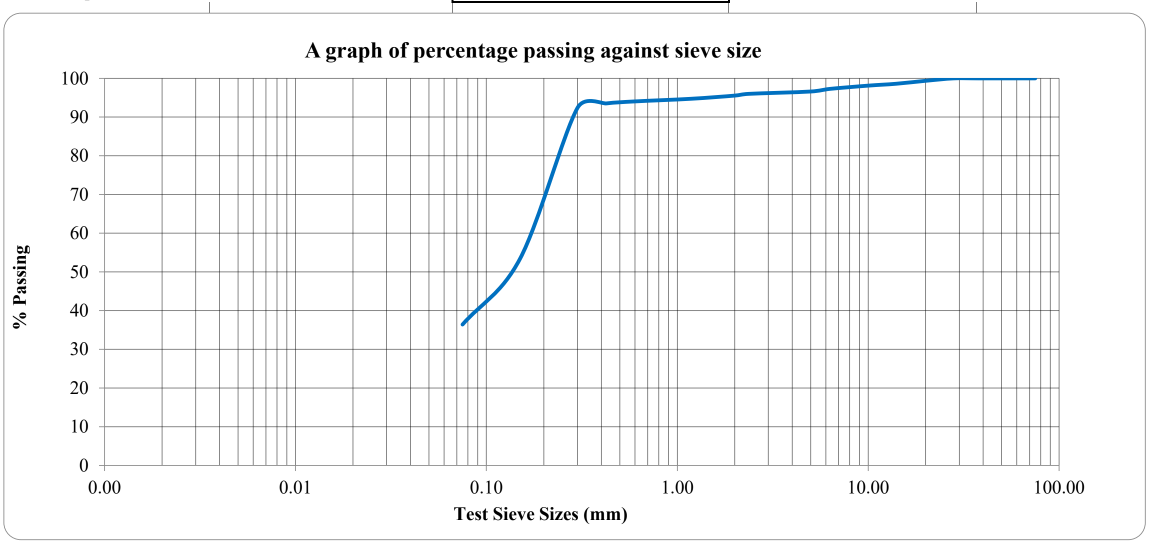
PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	Geotechnical Investigation For 400KV Karuma -Kawanda Transmission Line		
Client:	SAMUEL ACIDRI	**Depth (m)**	18.5-20.0
Location:	Kiryandongo	**Testing Date:**	
Sampling Date:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	AP 104/5	**Technician:**	Andrew / Farouk
Sample ref. No.:		**Checked By:**	EMMA
Soil Description:			
Initial wt before washing (g):	564.8	Moisture Content:	0.9
Dry wt after washing (g):	354.0	Initial Dry Weight (g):	559.5

Sieve Sizes (mm)	Partial Retained Mass (g)	Cumulative Retained Mass (g)	Cumulative Retained (%)	% Passing
75.0	0.0	0.0	0.0	100
50.0	0.0	0.0	0.0	100
37.5	0.0	0.0	0.0	100
28.0	0.0	0.0	0.0	100
20.0	3.5	3.5	0.6	99
14.0	4.3	7.8	1.4	99
10.0	2.7	10.5	1.9	98
6.3	4.7	15.2	2.7	97
5.0	3.8	19.0	3.4	97
2.36	3.4	22.4	4.0	96
2.0	2.5	24.9	4.5	96
1.18	4.6	29.5	5.3	95
0.60	3.8	33.3	6.0	94
0.425	2.9	36.2	6.5	94
0.300	6.5	42.7	7.6	92
0.150	218.1	260.8	46.6	53
0.075	95.0	355.8	63.6	36
pan	0.4	356.2		0.75



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
3.4	60.2	36.4

Remarks: These results relate to the sample that was tested

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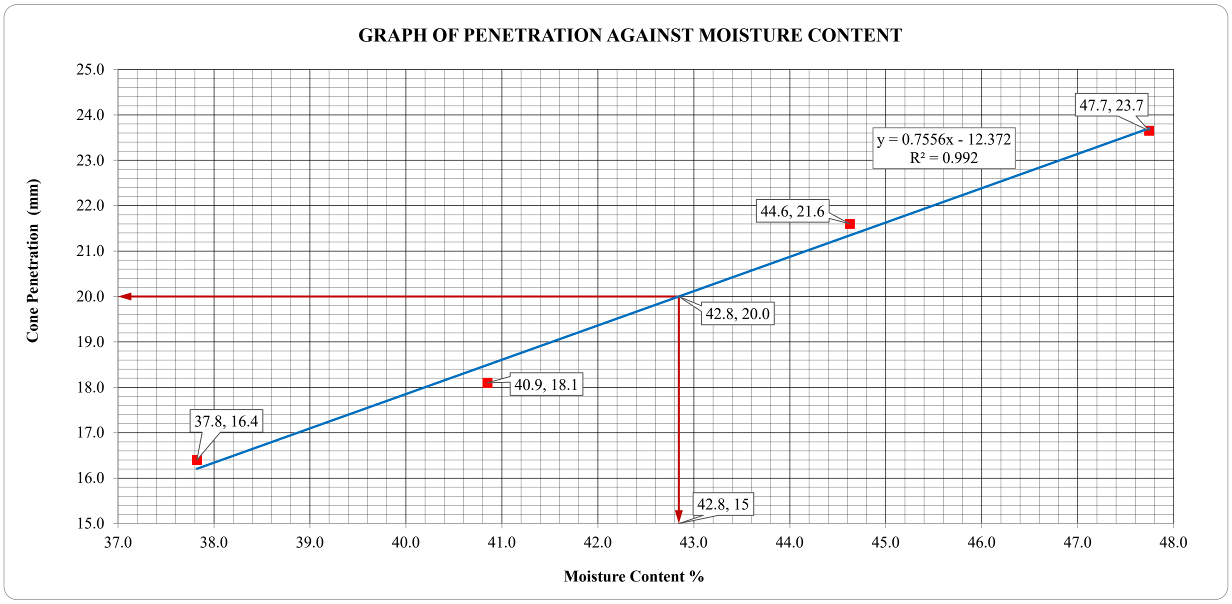
ATTERBERG LIMITS TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	1.0-3.5
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	EMMA

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		**PLASTIC LIMIT (%)**	
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	16.3	16.5	18.0	18.2	21.5	21.7	23.6	23.7		
Average Penetration	**mm**	16.4		18.1		21.6		23.7			
Container Number		**B3**	**Q3**	**H4**	**B2**	**X2**	**H7**	**E**	**W5**	**X2**	**MT**
Weight of wet soil + container:	**w2 (g)**	52.0	59.0	52.5	53.9	53.0	46.9	57.7	55.5	32.1	31.0
Weight of dry soil + container:	**w3 (g)**	46.2	51.1	46.1	46.9	46.1	40.1	49.1	47.3	29.6	28.8
Weight of container:	**w1 (g)**	30.8	30.3	30.4	29.8	30.6	24.9	30.9	30.3	16.7	17.5
Weight of moisture:	**w2 - w3 (g)**	5.8	7.9	6.4	7.0	6.9	6.8	8.6	8.2	2.5	2.2
Weight of dry soil:	**w3 - w1 (g)**	15.4	20.8	15.7	17.1	15.5	15.2	18.2	17.0	12.9	11.3
Moisture content:	**{(w2-w3) / (w3-w1)} *100**	37.7	38.0	40.8	40.9	44.5	44.7	47.3	48.2	19.4	19.5
Average Moisture Content:		37.8		40.9		44.6		47.7		19.4	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	131.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	6.4
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	42.84	**Plasticity Index (%)**	23.4
Plastic Limit (%)	19.42		

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS (U) LTD

Technical Manager

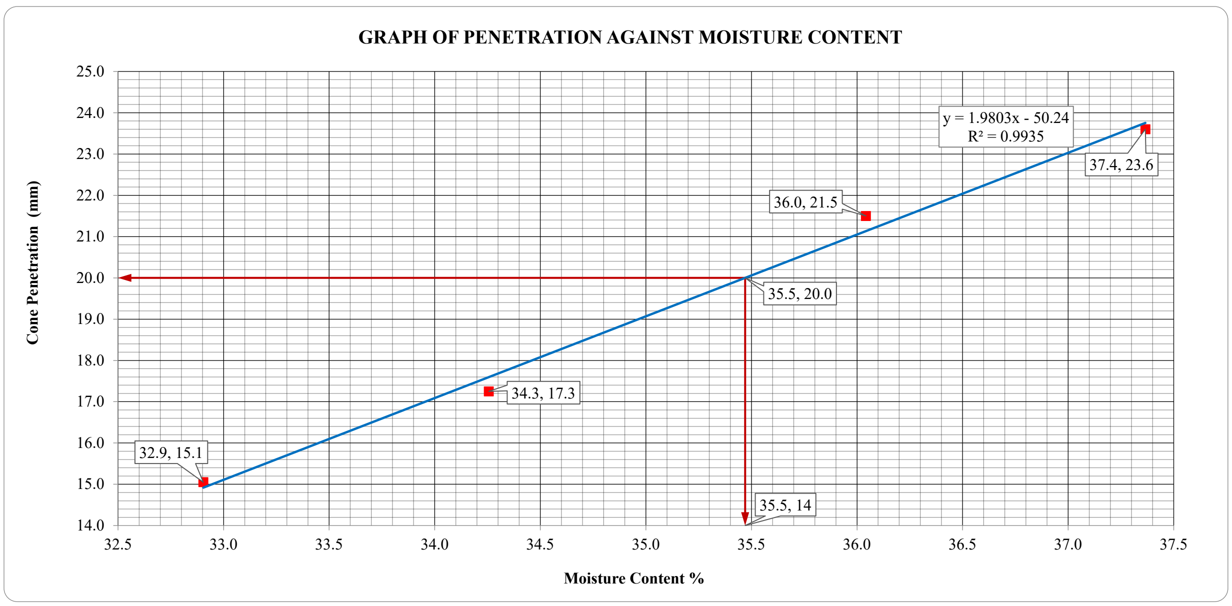
ATTERBERG LIMITS TEST REPORT

GEOTECH SOLUTIONS

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	3.5-5.0
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	EMMA

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		PLASTIC LIMIT (%)	
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	15.0	15.1	17.2	17.3	21.4	21.6	23.5	23.7		
Average Penetration	mm	15.1		17.3		21.5		23.6			
Container Number		O5	N6	U	D7	P3	T5	K7	U2	K12	K18
Weight of wet soil + container:	w2 (g)	40.2	41.5	46.9	45.3	48.1	45.8	42.0	43.6	31.8	32.4
Weight of dry soil + container:	w3 (g)	36.5	37.4	42.7	40.0	43.4	41.6	37.4	40.1	29.0	29.7
Weight of container:	w1 (g)	25.2	25.0	30.3	24.7	30.3	30.0	25.0	30.8	11.0	12.0
Weight of moisture:	w2 - w3 (g)	3.7	4.1	4.2	5.3	4.7	4.2	4.6	3.5	2.8	2.7
Weight of dry soil:	w3 - w1 (g)	11.3	12.4	12.4	15.3	13.1	11.6	12.4	9.3	18.0	17.7
Moisture content:	{(w2-w3) / (w3-w1)} *100	32.7	33.1	33.9	34.6	35.9	36.2	37.1	37.6	15.6	15.3
Average Moisture Content:		32.9		34.3		36.0		37.4		15.4	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	133.00	Linear Shrinkage $LS=100(1-L_D/L_o)$	5.0
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	35.5	Plasticity Index (%)	20.1
Plastic Limit (%)	15.4		

Remarks: These results relate to the sample that was tested

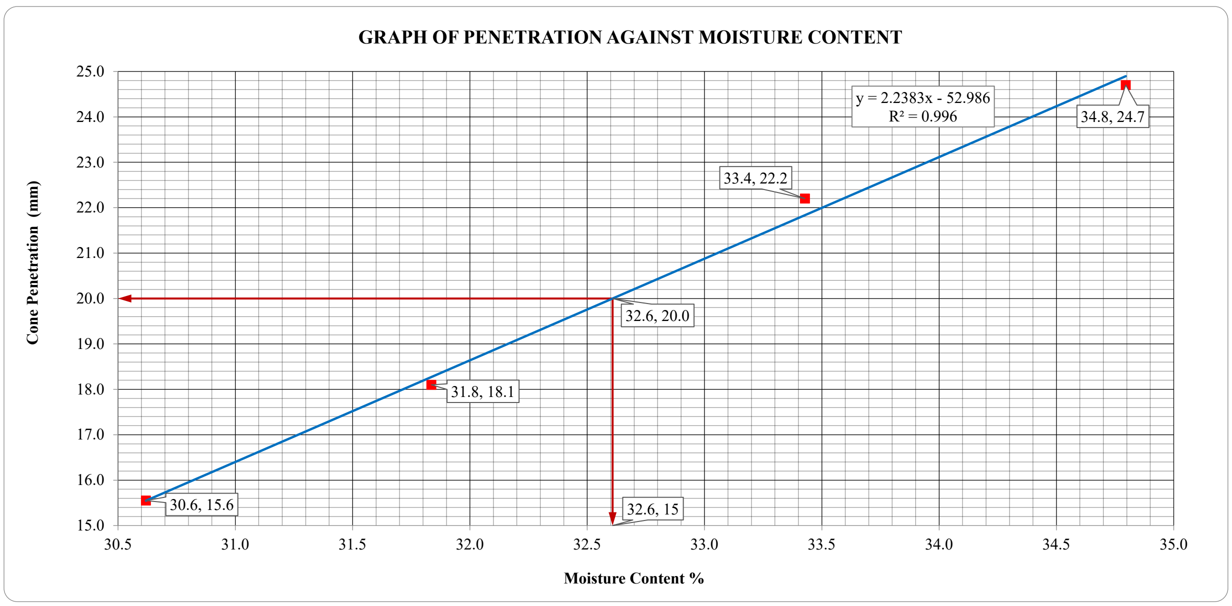
GEOTECH SOLUTIONS (U) LTD

Technical Manager

GEOTECH SOLUTIONS	ATTERBERG LIMITS TEST REPORT		
Project:	GEOTECHNICAL INVESTIGATIONS FOR 400KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	5.0-8.0
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	EMMA

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		PLASTIC LIMIT (%)	
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	15.5	15.6	18.0	18.2	22.1	22.3	24.6	24.8		
Average Penetration	**mm**	15.6		18.1		22.2		24.7			
Container Number		**X2**	**X1**	**X**	**KT**	**P6**	**TM**	**S3**	**W7**	**TO**	**ZY**
Weight of wet soil + container:	**w2 (g)**	41.3	46.5	57.9	47.9	46.8	40.2	43.7	43.1	34.1	31.1
Weight of dry soil + container:	**w3 (g)**	35.7	39.6	51.3	41.0	39.9	34.3	37.4	36.0	31.8	29.1
Weight of container:	**w1 (g)**	17.3	17.2	30.3	19.6	19.2	16.7	19.2	15.7	16.0	15.8
Weight of moisture:	**w2 - w3 (g)**	5.6	6.9	6.6	6.9	6.9	5.9	6.3	7.1	2.3	2.0
Weight of dry soil:	**w3 - w1 (g)**	18.4	22.4	21.0	21.4	20.7	17.6	18.2	20.3	15.8	13.3
Moisture content:	**{(w2-w3) / (w3-w1)} *100**	30.4	30.8	31.4	32.2	33.3	33.5	34.6	35.0	14.6	15.0
Average Moisture Content:		30.6		31.8		33.4		34.8		14.8	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	131.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	6.4
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	32.6	**Plasticity Index (%)**	17.8
Plastic Limit (%)	14.8		

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS (U) LTD

Technical Manager

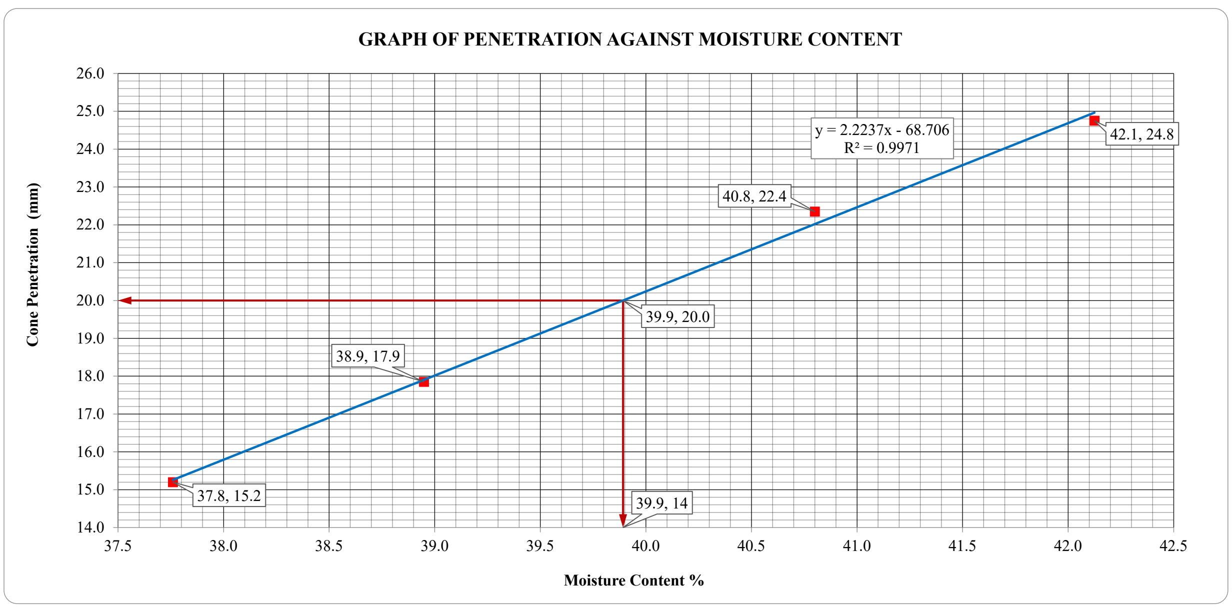
GEOTECH SOLUTIONS

ATTERBERG LIMITS TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	8.0-10.0
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	EMMA

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		**PLASTIC LIMIT (%)**	
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	15.1	15.3	17.8	17.9	22.2	22.5	24.7	24.8		
Average Penetration	**mm**	15.2		17.9		22.4		24.8			
Container Number		**V3**	**P1**	**T5**	**C1**	**F6**	**C**	**P3**	**H3**	**W5**	**B2**
Weight of wet soil + container:	**w2 (g)**	53.9	56.8	50.8	51.1	51.3	56.6	58.7	50.6	34.6	34.8
Weight of dry soil + container:	**w3 (g)**	47.4	49.4	43.6	45.3	43.6	49.0	50.3	43.0	31.3	31.1
Weight of container:	**w1 (g)**	30.1	29.9	25.0	30.5	24.7	30.4	30.2	25.1	17.4	16.2
Weight of moisture:	**w2 - w3 (g)**	6.5	7.4	7.2	5.8	7.7	7.6	8.4	7.6	3.3	3.7
Weight of dry soil:	**w3 - w1 (g)**	17.3	19.5	18.6	14.8	18.9	18.6	20.1	17.9	13.9	14.9
Moisture content:	**{(w2-w3) / (w3-w1)} *100**	37.6	37.9	38.7	39.2	40.7	40.9	41.8	42.5	23.7	24.8
Average Moisture Content:		37.8		38.9		40.8		42.1		24.3	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	128.00	**Linear Shrinkage $LS=100(1-L_D/L_o)$**	8.6
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	39.9	**Plasticity Index (%)**	15.6
Plastic Limit (%)	24.3		

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS (U) LTD

Technical Manager

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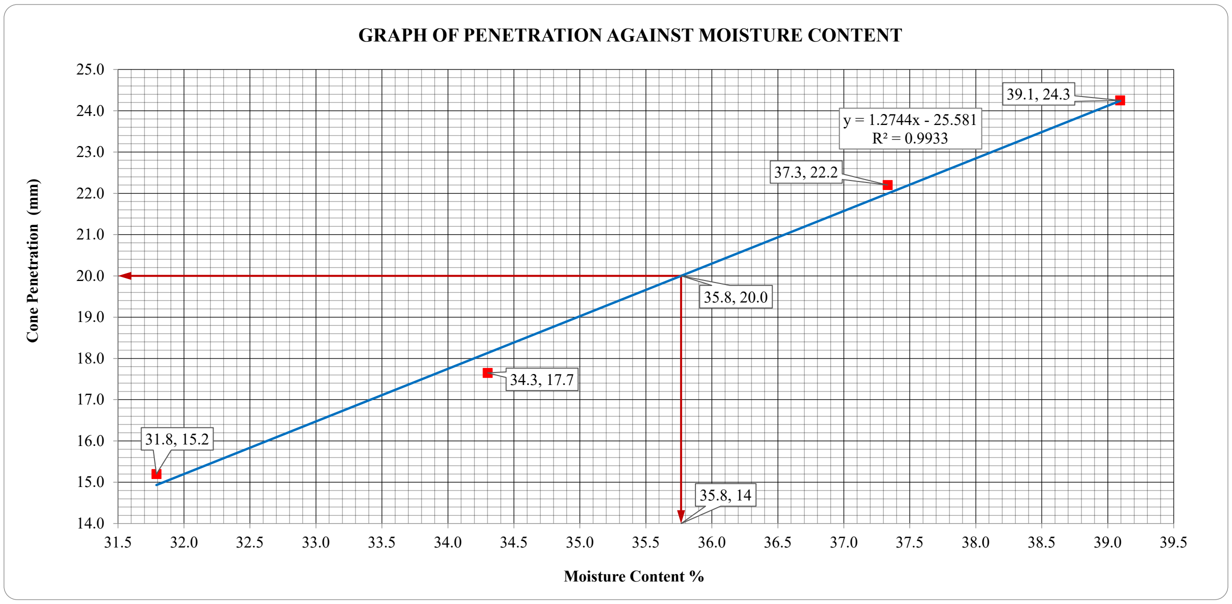
ATTERBERG LIMITS TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	10.0-12.0
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	EMMA

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		PLASTIC LIMIT (%)	
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	15.1	15.3	17.6	17.7	22.1	22.3	24.1	24.4		
Average Penetration	mm	15.2		17.7		22.2		24.3			
Container Number		BX	KIT	S10	CT	B17	BB	TY	BT	LR	ZB
Weight of wet soil + container:	w2 (g)	40.3	41.1	40.3	39.5	41.9	43.2	44.3	41.7	28.7	32.8
Weight of dry soil + container:	w3 (g)	34.7	35.8	34.2	33.7	35.0	36.1	36.9	34.6	26.6	30.3
Weight of container:	w1 (g)	16.9	19.3	16.3	16.9	16.3	17.3	17.8	16.6	15.5	17.2
Weight of moisture:	w2 - w3 (g)	5.6	5.3	6.1	5.8	6.9	7.1	7.4	7.1	2.1	2.5
Weight of dry soil:	w3 - w1 (g)	17.8	16.5	17.9	16.8	18.7	18.8	19.1	18.0	11.1	13.1
Moisture content:	{(w2-w3) / (w3-w1)} *100	31.5	32.1	34.1	34.5	36.9	37.8	38.7	39.4	18.9	19.1
Average Moisture Content:		31.8		34.3		37.3		39.1		19.0	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	131.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	6.4
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	35.8	**Plasticity Index (%)**	16.8
Plastic Limit (%)	19.0		

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS (U) LTD

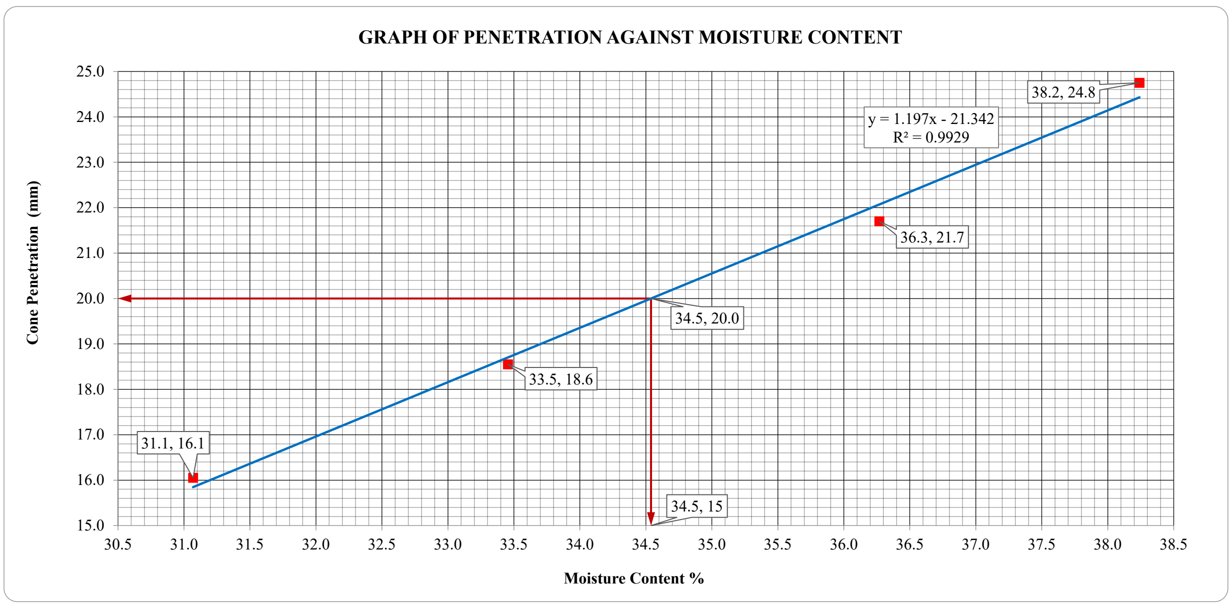
Technical Manager

ATTERBERG LIMITS TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400 KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	12.0-15.0
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	Emma

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		**1**		**2**		**3**		**4**		**PLASTIC LIMIT (%)**	
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	16.0	16.1	18.5	18.6	21.6	21.8	24.7	24.8		
Average Penetration	**mm**	16.1		18.6		21.7		24.8			
Container Number		**G6**	**E1**	**G3**	**V5**	**S4**	**M2**	**W5**	**X2**	**NA**	**TK**
Weight of wet soil + container:	**w2 (g)**	47.3	45.8	45.6	48.5	44.6	43.4	49.2	47.5	34.7	35.0
Weight of dry soil + container:	**w3 (g)**	43.6	42.2	41.8	43.9	40.9	40.1	44.0	42.8	32.6	32.8
Weight of container:	**w1 (g)**	31.6	30.7	30.4	30.2	30.7	31.0	30.3	30.6	18.6	17.8
Weight of moisture:	**w2 - w3 (g)**	3.7	3.6	3.8	4.6	3.7	3.3	5.2	4.7	2.1	2.2
Weight of dry soil:	**w3 - w1 (g)**	12.0	11.5	11.4	13.7	10.2	9.1	13.7	12.2	14.0	15.0
Moisture content:	**{(w2-w3) / (w3-w1)} *100**	30.8	31.3	33.3	33.6	36.3	36.3	38.0	38.5	15.0	14.7
Average Moisture Content:		31.1		33.5		36.3		38.2		14.8	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	133.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	5.0
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	34.5	**Plasticity Index (%)**	19.7
Plastic Limit (%)	14.8		

Remarks: These results relate to the sample that was tested

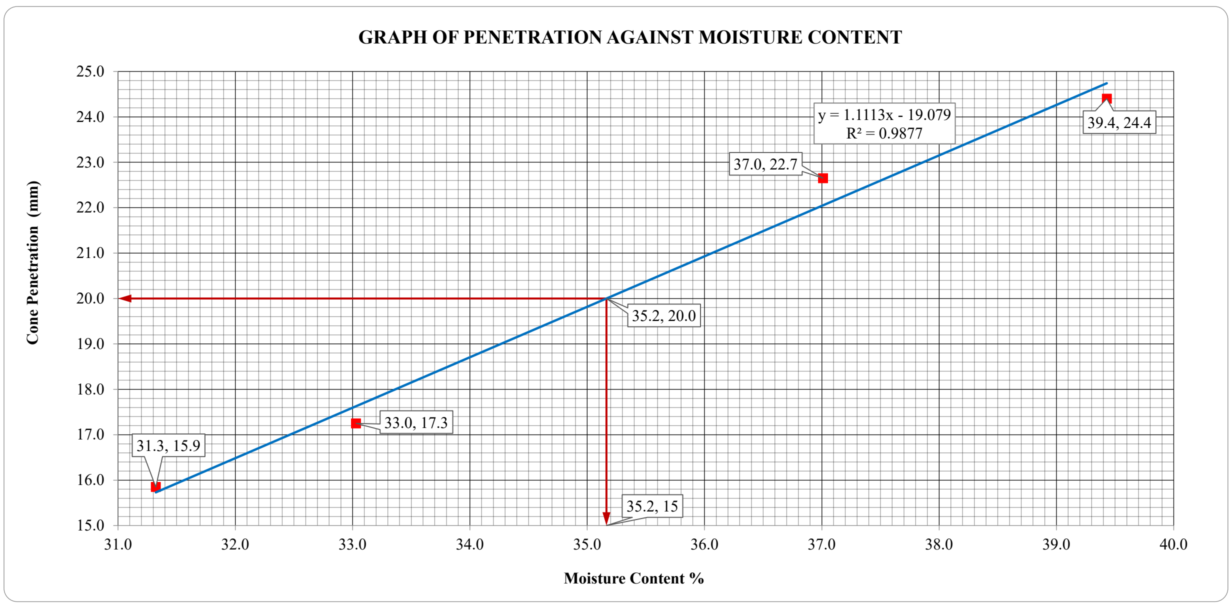
GEOTECH SOLUTIONS (U) LTD

Technical Manager

GEOTECH SOLUTIONS	ATTERBERG LIMITS TEST REPORT		
Project:	GEOTECHNICAL INVESTIGATIONS FOR 400 KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	15.0-16.3
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	Emma

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		PLASTIC LIMIT (%)	
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	15.8	15.9	17.2	17.3	22.6	22.7	24.3	24.5		
Average Penetration	**mm**	15.9		17.3		22.7		24.4			
Container Number		**W10**	**ZA**	**Z2**	**W7**	**Z3**	**W4**	**W9**	**X10**	**K21**	**BZ**
Weight of wet soil + container:	**w2 (g)**	40.9	52.2	35.7	46.5	51.8	51.4	50.9	54.7	25.7	30.0
Weight of dry soil + container:	**w3 (g)**	34.3	45.9	30.1	41.4	44.2	44.2	43.4	46.9	23.6	27.8
Weight of container:	**w1 (g)**	13.0	26.0	13.1	26.0	23.5	24.9	24.2	27.3	12.2	16.1
Weight of moisture:	**w2 - w3 (g)**	6.6	6.3	5.6	5.1	7.6	7.2	7.5	7.8	2.1	2.2
Weight of dry soil:	**w3 - w1 (g)**	21.3	19.9	17.0	15.4	20.7	19.3	19.2	19.6	11.4	11.7
Moisture content:	**{(w2-w3) / (w3-w1)} *100**	31.0	31.7	32.9	33.1	36.7	37.3	39.1	39.8	18.4	18.8
Average Moisture Content:		31.3		33.0		37.0		39.4		18.6	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	128.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	8.6
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	35.2	**Plasticity Index (%)**	16.6
Plastic Limit (%)	18.6		

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS (U) LTD

Technical Manager

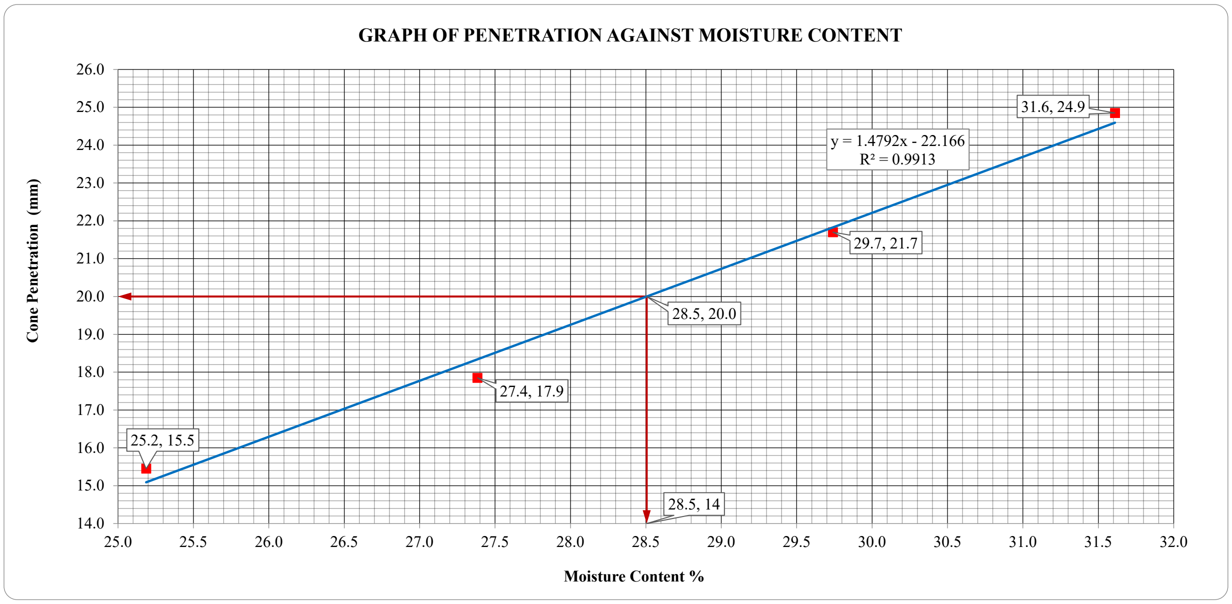
GEOTECH SOLUTIONS

ATTERBERG LIMITS TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	16.3-18.0
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	EMMA

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		1		2		3		4		PLASTIC LIMIT (%)
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
Final dial gauge reading	mm	15.4	15.5	17.8	17.9	21.6	21.8	24.8	24.9	
Average Penetration	mm	15.5		17.9		21.7		24.9		
Container Number		G	T1	J	M2	U2	M1	Q1	Z3	NP
Weight of wet soil + container:	w2 (g)	56.3	55.8	56.3	57.7	58.6	57.2	53.0	61.2	
Weight of dry soil + container:	w3 (g)	51.3	50.7	50.8	52.0	52.2	51.2	46.2	53.9	
Weight of container:	w1 (g)	31.4	30.5	30.7	31.2	30.6	31.1	24.6	30.9	
Weight of moisture:	w2 - w3 (g)	5.0	5.1	5.5	5.7	6.4	6.0	6.8	7.3	
Weight of dry soil:	w3 - w1 (g)	19.9	20.2	20.1	20.8	21.6	20.1	21.6	23.0	
Moisture content:	{(w2-w3) / (w3-w1)} *100	25.1	25.2	27.4	27.4	29.6	29.9	31.5	31.7	
Average Moisture Content:		25.2		27.4		29.7		31.6		



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	128.00	Linear Shrinkage LS=100(1-L_D/L_o)	8.6
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	28.5	Plasticity Index (%)	NP
Plastic Limit (%)	NP		

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS (U) LTD

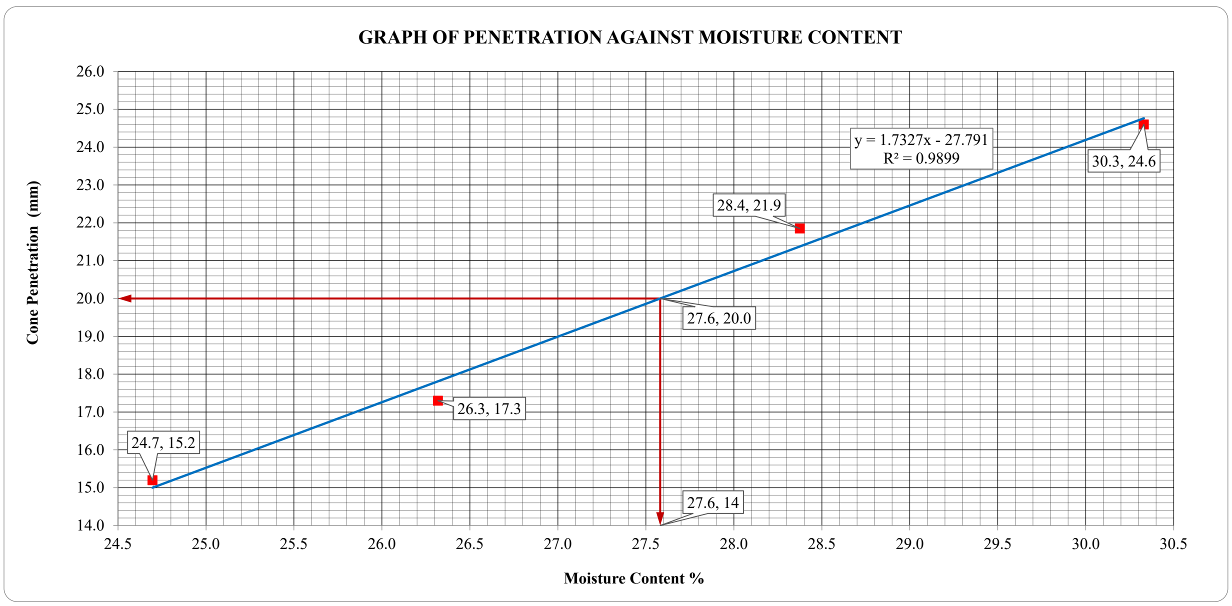
Technical Manager

ATTERBERG LIMITS TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS FOR 400KV KARUMA-KAWANDA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI		
Location:	Kiryandongo	**Depth (m):**	18.0-20.0
Sampling Date:		**Testing date:**	
BH/ Pit No.:	AP 104/5	**Test method:**	BS 1377: Part 2: 1990
Sample ref. No.:		**Technician:**	HENRY/HARRIET
Soil Description:		**Checked By:**	EMMA

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD

TEST No.		**1**		**2**		**3**		**4**		
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	**PLASTIC LIMIT (%)**
Final dial gauge reading	**mm**	15.1	15.3	17.2	17.4	21.8	21.9	24.5	24.7	
Average Penetration	**mm**	15.2		17.3		21.9		24.6		
Container Number		**E1**	**V5**	**I4**	**G3**	**N6**	**Y3**	**D7**	**S4**	
Weight of wet soil + container:	**w2 (g)**	56.6	54.8	49.7	53.4	53.2	59.2	52.1	58.4	
Weight of dry soil + container:	**w3 (g)**	51.5	49.9	44.5	48.6	47.0	52.6	45.8	51.9	
Weight of container:	**w1 (g)**	30.7	30.2	24.7	30.4	24.9	29.6	24.8	30.7	**NP**
Weight of moisture:	**w2 - w3 (g)**	5.1	4.9	5.2	4.8	6.2	6.6	6.3	6.5	
Weight of dry soil:	**w3 - w1 (g)**	20.8	19.7	19.8	18.2	22.1	23.0	21.0	21.2	
Moisture content:	**{(w2-w3) / (w3-w1)} *100**	24.5	24.9	26.3	26.4	28.1	28.7	30.0	30.7	
Average Moisture Content:		24.7		26.3		28.4		30.3		



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	133.00	**Linear Shrinkage LS=100(1-L_D/L_o)**	5.0
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit (%)	27.6	**Plasticity Index (%)**	NP
Plastic Limit (%)	NP		

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS (U) LTD

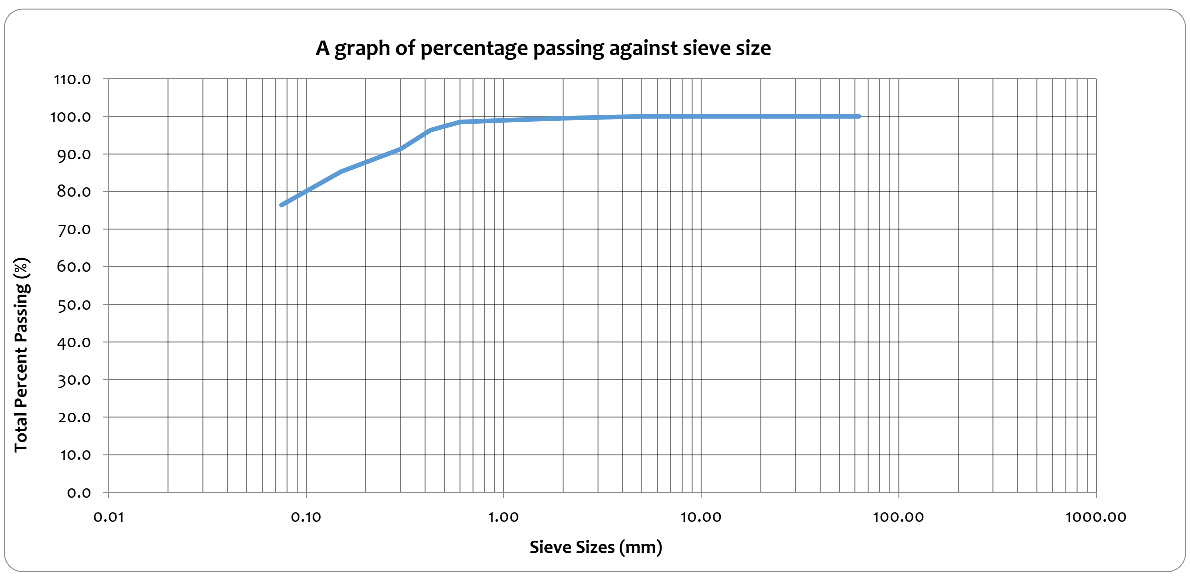
Technical Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m)**	0.00 - 0.4
Location:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	KL 30 (B103+5)	**Technician:**	Eddy Watema

Dry mass before washing (A)- gm:	1276.2	Dry mass after washing (B)- gm:	326.3	C = A - B (gm)
Mass of dry sample (gm), m_1:	1087.5	Moisture Content (%):	23.7	949.9

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulattive % Passing
63.000	0.0	0.0	0.0	0.0	100.0	100.0
50.000	0.0	0.0	0.0	0.0	100.0	100.0
37.500	0.0	0.0	0.0	0.0	100.0	100.0
28.000	0.0	0.0	0.0	0.0	100.0	100.0
20.000	0.0	0.0	0.0	0.0	100.0	100.0
14.000	0.0	0.0	0.0	0.0	100.0	100.0
10.000	0.0	0.0	0.0	0.0	100.0	100.0
6.300	0.0	0.0	0.0	0.0	100.0	100.0
5.000	0.0	0.0	0.0	0.0	100.0	100.0
2.000	5.4	5.4	0.5	0.5	99.5	99.5
1.180	4.4	9.8	0.4	0.9	99.6	99.1
0.600	6.5	16.3	0.6	1.5	99.4	98.5
0.425	23.9	40.2	2.2	3.7	97.8	96.3
0.300	54.4	94.6	5.0	8.7	95.0	91.3
0.150	65.3	159.9	6.0	14.7	94.0	85.3
0.075	96.8	256.7	8.9	23.6	91.1	76.4
Pan	0.0	256.7				
Pan + C	949.9	1206.6				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
0.5	23.1	76.4

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

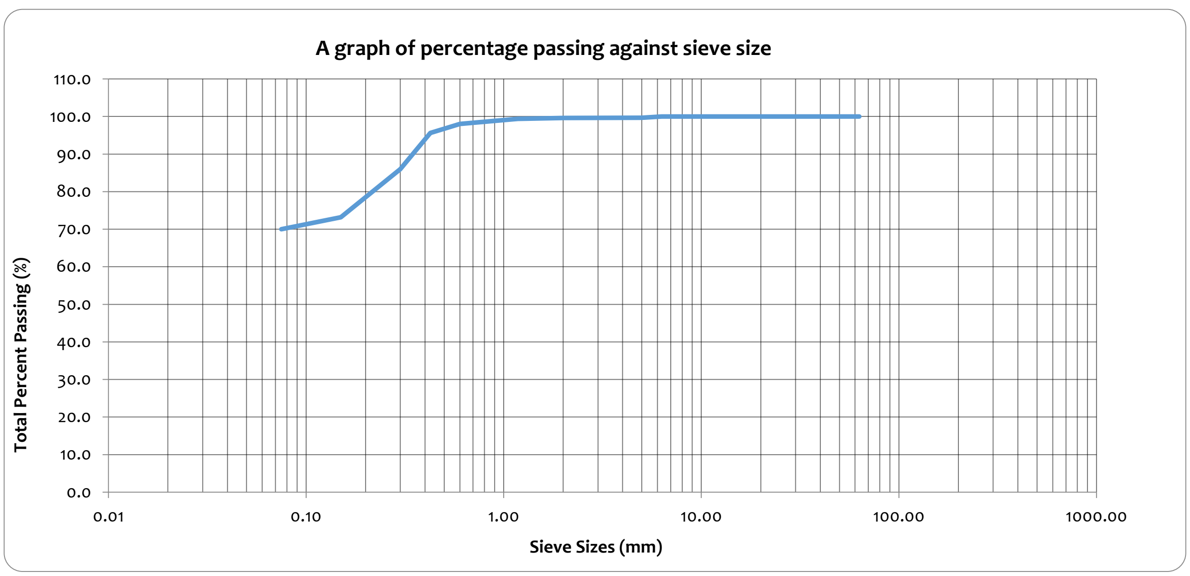
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m)**	0.4 - 1.5
Location:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	KL 30 (B103+5)	**Technician:**	Eddy Watema

Dry mass before washing (A)- gm:	981.0	Dry mass after washing (B)- gm:	239.9	C = A - B (gm)
Mass of dry sample (gm), m_1:	792.7	Moisture Content (%):	23.7	741.1

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	0.0	0.0	0.0	0.0	100.0	100.0
50.000	0.0	0.0	0.0	0.0	100.0	100.0
37.500	0.0	0.0	0.0	0.0	100.0	100.0
28.000	0.0	0.0	0.0	0.0	100.0	100.0
20.000	0.0	0.0	0.0	0.0	100.0	100.0
14.000	0.0	0.0	0.0	0.0	100.0	100.0
10.000	0.0	0.0	0.0	0.0	100.0	100.0
6.300	0.0	0.0	0.0	0.0	100.0	100.0
5.000	2.7	2.7	0.3	0.3	99.7	99.7
2.000	0.5	3.2	0.1	0.4	99.9	99.6
1.180	1.8	5.0	0.2	0.6	99.8	99.4
0.600	10.5	15.5	1.3	2.0	98.7	98.0
0.425	19.3	34.8	2.4	4.4	97.6	95.6
0.300	76.4	111.2	9.6	14.0	90.4	86.0
0.150	101.4	212.6	12.8	26.8	87.2	73.2
0.075	25.2	237.8	3.2	30.0	96.8	70.0
Pan	2.1	239.9				
Pan + C	743.2	983.1				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
0.4	29.6	70.0

Remarks: These results relate to the sample that was tested

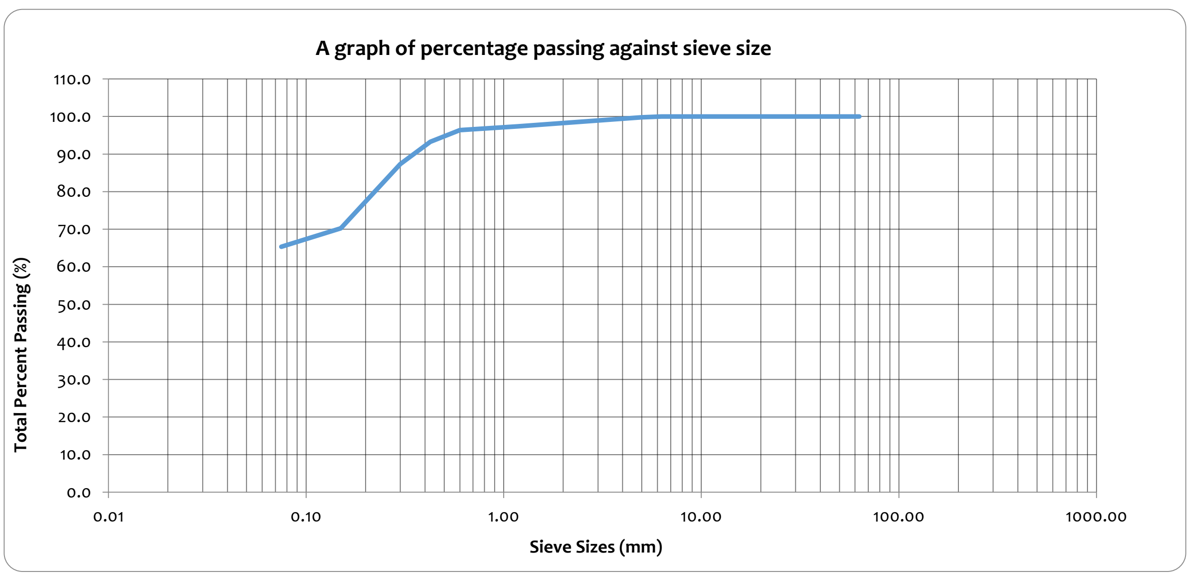
GEOTECH SOLUTIONS

Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE					
Client:	SAMUEL ACIDRI		**Depth (m)**		1.5 - 2.0	
Location:			**Test Method:**		BS 1377: Part 2: 1990	
Pit No.:	KL 30 (B103+5)		**Technician:**		Eddy Watema	
Dry mass before washing (A)- gm:		1752.3	Dry mass after washing (B)- gm:		488.6	C = A - B (gm)
Mass of dry sample (gm), m_1:		1409.4	Moisture Content (%):		24.3	1263.7

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	0.00	0.0	0.0	0.0	100.0	100.0
14.000	0.00	0.0	0.0	0.0	100.0	100.0
10.000	0.00	0.0	0.0	0.0	100.0	100.0
6.300	0.00	0.0	0.0	0.0	100.0	100.0
5.000	2.80	2.8	0.2	0.2	99.8	99.8
2.000	21.81	24.6	1.5	1.7	98.5	98.3
1.180	12.50	37.1	0.9	2.6	99.1	97.4
0.600	14.10	51.2	1.0	3.6	99.0	96.4
0.425	43.70	94.9	3.1	6.7	96.9	93.3
0.300	83.20	178.1	5.9	12.6	94.1	87.4
0.150	241.60	419.7	17.1	29.8	82.9	70.2
0.075	68.90	488.6	4.9	34.7	95.1	65.3
Pan	0.00	488.6				
Pan + C	1263.70	1752.3				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
1.75	32.9	65.3

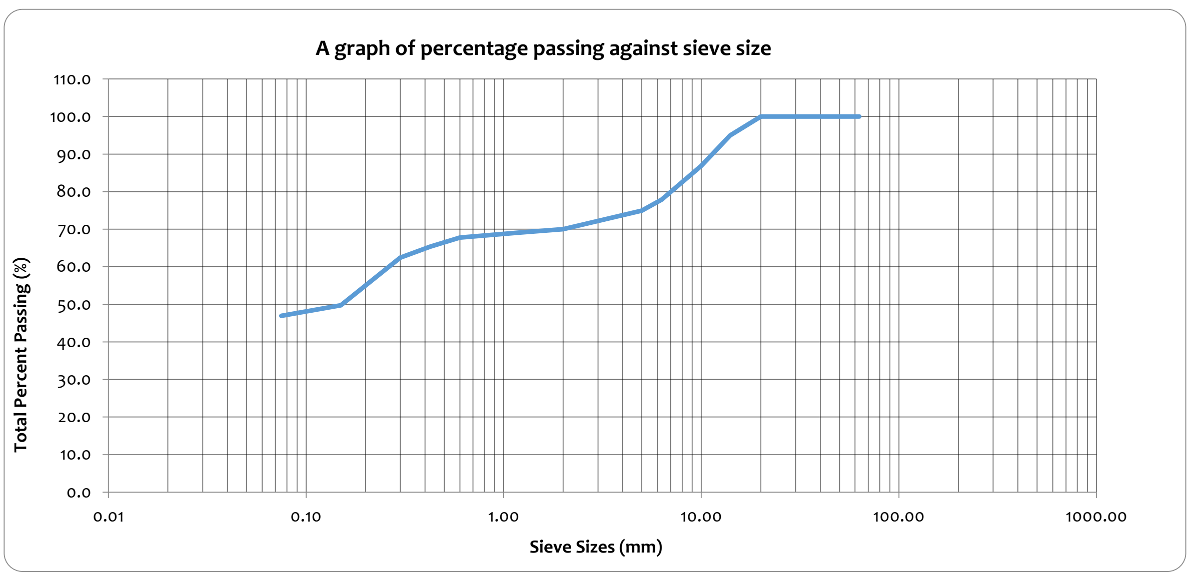
Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE					
Client:	SAMUEL ACIDRI		**Depth (m)**		2.0 - 3.0	
Location:			**Test Method:**		BS 1377: Part 2: 1990	
Pit No.:	KL 30 (B103+5)		**Technician:**		Eddy Watema	
Dry mass before washing (A)- gm:		952.00	Dry mass after washing (B)- gm:		434.90	C = A - B (gm)
Mass of dry sample (gm), m_1:		816.90	Moisture Content (%):		16.50	517.10
Sieve Sizes (mm)	**Mass Retained Mass (g)**	**Cumulative Retained Mass (g)**	**% Retained**	**Cumulative Retained (%)**	**% Passing**	**Cumulative % Passing**
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	0.00	0.0	0.0	0.0	100.0	100.0
14.000	40.90	40.9	5.0	5.0	95.0	95.0
10.000	66.20	107.1	8.1	13.1	91.9	86.9
6.300	73.80	180.9	9.0	22.1	91.0	77.9
5.000	23.70	204.6	2.9	25.0	97.1	75.0
2.000	40.60	245.2	5.0	30.0	95.0	70.0
1.180	7.50	252.7	0.9	30.9	99.1	69.1
0.600	10.60	263.3	1.3	32.2	98.7	67.8
0.425	19.70	283.0	2.4	34.6	97.6	65.4
0.300	24.00	307.0	2.9	37.6	97.1	62.4
0.150	103.30	410.3	12.6	50.2	87.4	49.8
0.075	23.10	433.4	2.8	53.1	97.2	46.9
Pan	1.50	434.9				
Pan + C	518.60	953.5				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
30.02	23.0	46.9

Remarks: These results relate to the sample that was tested

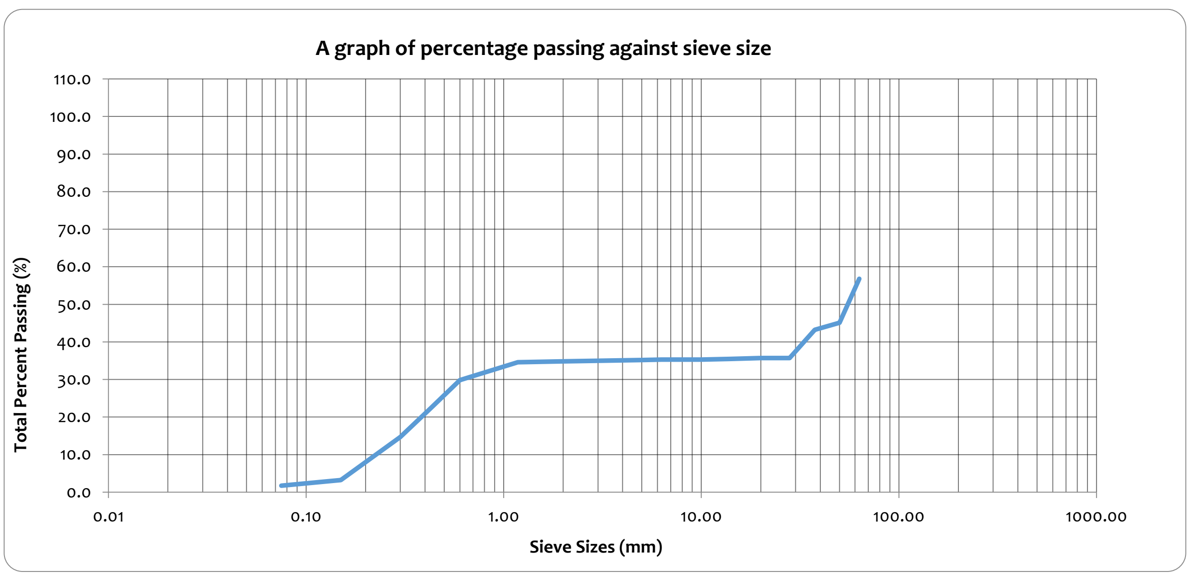
GEOTECH SOLUTIONS

Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE			
Client:	SAMUEL ACIDRI	**Depth (m)**	3.0 - 4.50	
Location:		**Test Method:**	BS 1377: Part 2: 1990	
Pit No.:	KL 30 (B103+5)	**Technician:**	Eddy Watema	
Dry mass before washing (A)- gm:	1532.40	Dry mass after washing (B)- gm:	1338.50	C = A - B (gm)
Mass of dry sample (gm), m_1:	1362.10	Moisture Content (%):	12.50	193.90

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	588.40	588.4	43.2	43.2	56.8	56.8
50.000	159.10	747.5	11.7	54.9	88.3	45.1
37.500	26.20	773.7	1.9	56.8	98.1	43.2
28.000	101.70	875.4	7.5	64.3	92.5	35.7
20.000	0.00	875.4	0.0	64.3	100.0	35.7
14.000	3.50	878.9	0.3	64.5	99.7	35.5
10.000	2.60	881.5	0.2	64.7	99.8	35.3
6.300	0.20	881.7	0.0	64.7	100.0	35.3
5.000	1.10	882.8	0.1	64.8	99.9	35.2
2.000	4.70	887.5	0.3	65.2	99.7	34.8
1.180	3.10	890.6	0.2	65.4	99.8	34.6
0.600	64.80	955.4	4.8	70.1	95.2	29.9
0.425	103.30	1058.7	7.6	77.7	92.4	22.3
0.300	103.40	1162.1	7.6	85.3	92.4	14.7
0.150	155.90	1318.0	11.4	96.8	88.6	3.2
0.075	20.50	1338.5	1.5	98.3	98.5	1.7
Pan	0.00	1338.5				
Pan + C	193.90	1532.4				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
65.16	33.1	1.7

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

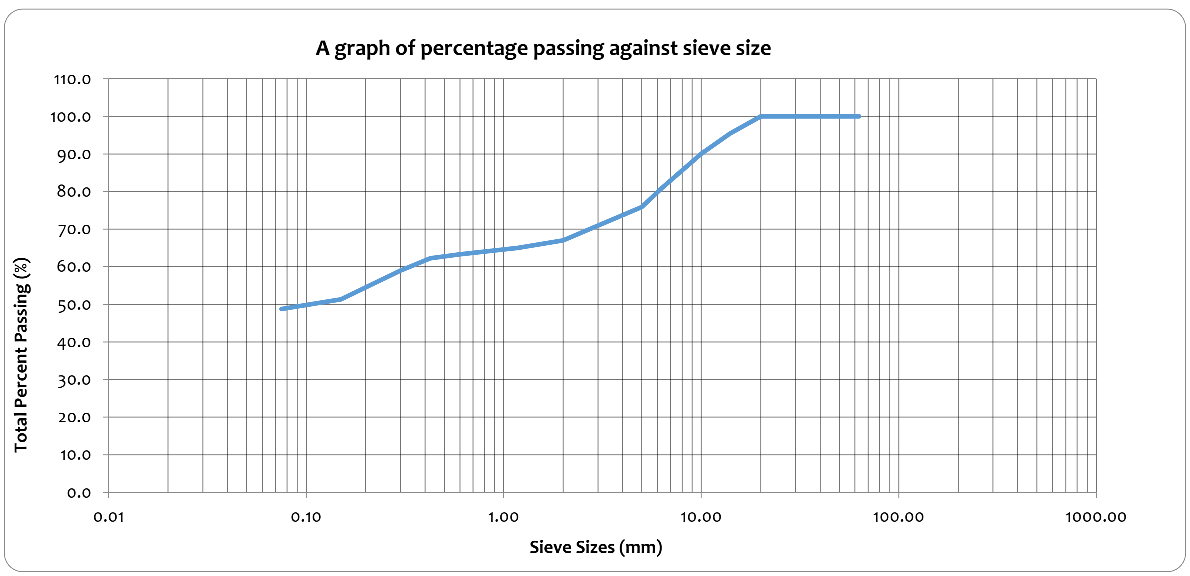
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m)**	4.5 - 5.1
Location:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	KL 30 (B103+5)	**Technician:**	Eddy Watema

Dry mass before washing (A)- gm:	1044.30	Dry mass after washing (B)- gm:	451.60	C = A - B (gm)
Mass of dry sample (gm), m_1:	880.40	Moisture Content (%):	18.60	592.70

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	0.00	0.0	0.0	0.0	100.0	100.0
14.000	40.10	40.1	4.6	4.6	95.4	95.4
10.000	47.70	87.8	5.4	10.0	94.6	90.0
6.300	80.60	168.4	9.2	19.1	90.8	80.9
5.000	43.80	212.2	5.0	24.1	95.0	75.9
2.000	78.30	290.5	8.9	33.0	91.1	67.0
1.180	17.40	307.9	2.0	35.0	98.0	65.0
0.600	14.90	322.8	1.7	36.7	98.3	63.3
0.425	9.50	332.3	1.1	37.7	98.9	62.3
0.300	29.20	361.5	3.3	41.1	96.7	58.9
0.150	67.20	428.7	7.6	48.7	92.4	51.3
0.075	22.40	451.1	2.5	51.2	97.5	48.8
Pan	0.50	451.6				
Pan + C	593.20	1044.8				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
33.00	18.2	48.8

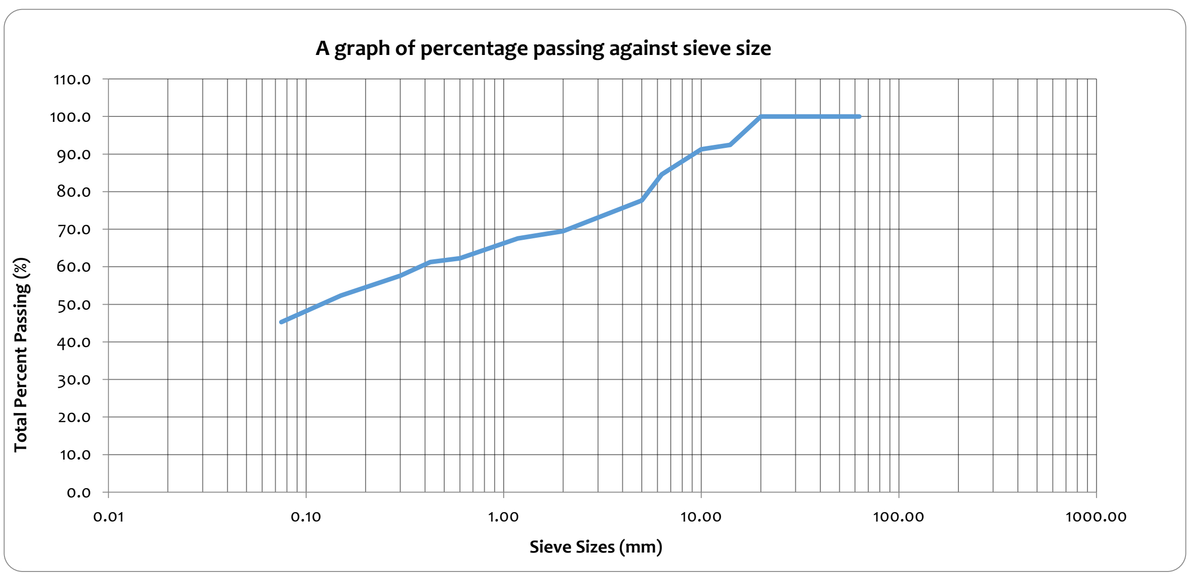
Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE					
Client:	SAMUEL ACIDRI		**Depth (m)**		5.1 - 6.0	
Location:			**Test Method:**		BS 1377: Part 2: 1990	
Pit No.:	KL 30 (B103+5)		**Technician:**		Eddy Watema	
Dry mass before washing (A)- gm:		1431.20	Dry mass after washing (B)- gm:		654.80	C = A - B (gm)
Mass of dry sample (gm), m_1:		1196.40	Moisture Content (%):		19.60	776.40
Sieve Sizes (mm)	**Mass Retained Mass (g)**	**Cumulative Retained Mass (g)**	**% Retained**	**Cumulative Retained (%)**	**% Passing**	**Cumulative % Passing**
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	0.00	0.0	0.0	0.0	100.0	100.0
14.000	90.30	90.3	7.5	7.5	92.5	92.5
10.000	14.30	104.6	1.2	8.7	98.8	91.3
6.300	80.10	184.7	6.7	15.4	93.3	84.6
5.000	82.60	267.3	6.9	22.3	93.1	77.7
2.000	98.10	365.4	8.2	30.5	91.8	69.5
1.180	22.60	388.0	1.9	32.4	98.1	67.6
0.600	63.90	451.9	5.3	37.8	94.7	62.2
0.425	11.80	463.7	1.0	38.8	99.0	61.2
0.300	43.20	506.9	3.6	42.4	96.4	57.6
0.150	63.10	570.0	5.3	47.6	94.7	52.4
0.075	84.70	654.7	7.1	54.7	92.9	45.3
Pan	0.00	654.7				
Pan + C	776.40	1431.1				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
30.54	24.2	45.3

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

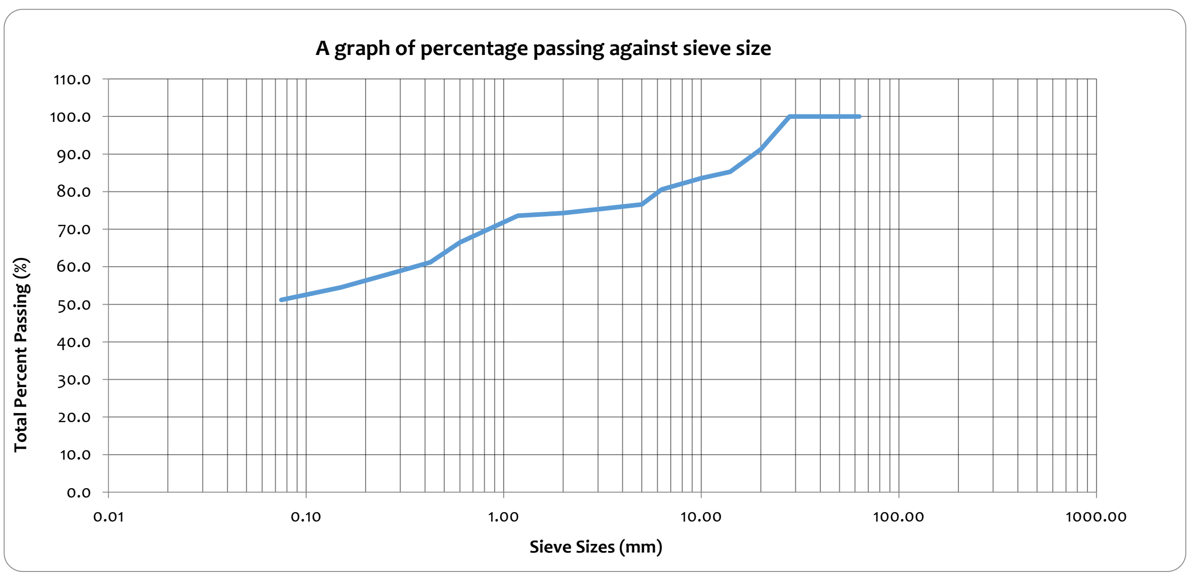
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m)**	6.0 - 7.0
Location:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	KL 30 (B103+5)	**Technician:**	Eddy Watema

Dry mass before washing (A)- gm:	1346.80	Dry mass after washing (B)- gm:	542.30	C = A - B (gm)
Mass of dry sample (gm), m_1:	1111.30	Moisture Content (%):	21.20	804.50

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	96.70	96.7	8.7	8.7	91.3	91.3
14.000	66.70	163.4	6.0	14.7	94.0	85.3
10.000	18.90	182.3	1.7	16.4	98.3	83.6
6.300	33.30	215.6	3.0	19.4	97.0	80.6
5.000	44.50	260.1	4.0	23.4	96.0	76.6
2.000	25.60	285.7	2.3	25.7	97.7	74.3
1.180	7.80	293.5	0.7	26.4	99.3	73.6
0.600	78.90	372.4	7.1	33.5	92.9	66.5
0.425	58.90	431.3	5.3	38.8	94.7	61.2
0.300	25.60	456.9	2.3	41.1	97.7	58.9
0.150	48.90	505.8	4.4	45.5	95.6	54.5
0.075	36.70	542.5	3.3	48.8	96.7	51.2
Pan	0.00	542.5				
Pan + C	804.50	1347.0				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
25.71	23.1	51.2

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

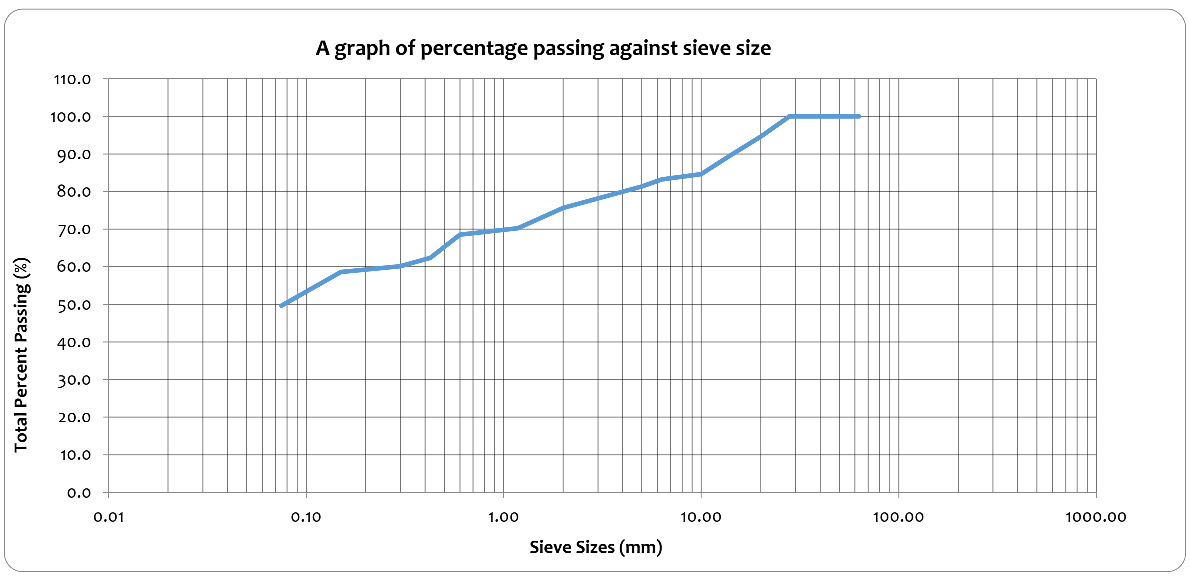
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m)**	7.0 - 7.8
Location:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	KL 30 (B103+5)	**Technician:**	Eddy Watema

Dry mass before washing (A)- gm:	2131.50	Dry mass after washing (B)- gm:	893.50	C = A - B (gm)
Mass of dry sample (gm), m_1:	1774.60	Moisture Content (%):	20.10	1238.00

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	95.30	95.3	5.4	5.4	94.6	94.6
14.000	90.10	185.4	5.1	10.4	94.9	89.6
10.000	87.40	272.8	4.9	15.4	95.1	84.6
6.300	24.60	297.4	1.4	16.8	98.6	83.2
5.000	33.40	330.8	1.9	18.6	98.1	81.4
2.000	101.70	432.5	5.7	24.4	94.3	75.6
1.180	95.40	527.9	5.4	29.7	94.6	70.3
0.600	30.00	557.9	1.7	31.4	98.3	68.6
0.425	110.20	668.1	6.2	37.6	93.8	62.4
0.300	39.60	707.7	2.2	39.9	97.8	60.1
0.150	26.40	734.1	1.5	41.4	98.5	58.6
0.075	159.40	893.5	9.0	50.3	91.0	49.7
Pan	0.00	893.5				
Pan + C	1238.00	2131.5				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
24.37	26.0	49.7

Remarks: These results relate to the sample that was tested

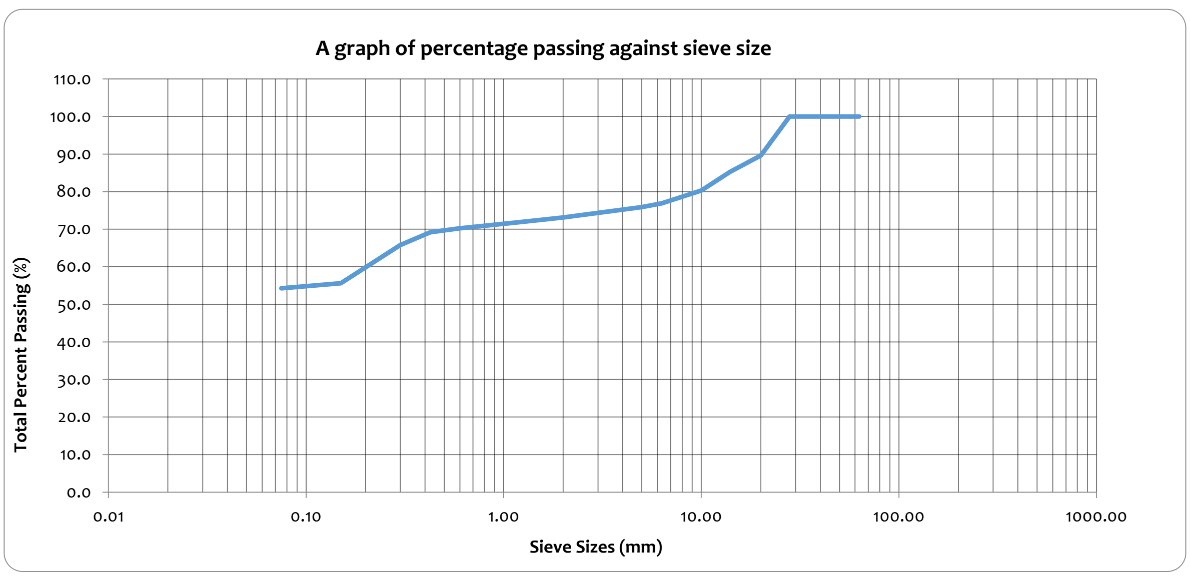
GEOTECHNICAL SOLUTIONS

Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE					
Client:	SAMUEL ACIDRI		**Depth (m)**		7.8 - 9.0	
Location:			**Test Method:**		BS 1377: Part 2: 1990	
Pit No.:	KL 30 (B103+5)		**Technician:**		Eddy Watema	
Dry mass before washing (A)- gm:		1086.00	Dry mass after washing (B)- gm:		429.60	C = A - B (gm)
Mass of dry sample (gm), m_1:		938.40	Moisture Content (%):		15.70	656.40

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	97.50	97.5	10.4	10.4	89.6	89.6
14.000	40.40	137.9	4.3	14.7	95.7	85.3
10.000	47.10	185.0	5.0	19.7	95.0	80.3
6.300	32.10	217.1	3.4	23.1	96.6	76.9
5.000	9.40	226.5	1.0	24.1	99.0	75.9
2.000	25.90	252.4	2.8	26.9	97.2	73.1
1.180	11.90	264.3	1.3	28.2	98.7	71.8
0.600	14.70	279.0	1.6	29.7	98.4	70.3
0.425	10.20	289.2	1.1	30.8	98.9	69.2
0.300	32.10	321.3	3.4	34.2	96.6	65.8
0.150	95.20	416.5	10.1	44.4	89.9	55.6
0.075	12.60	429.1	1.3	45.7	98.7	54.3
Pan	0.50	429.6				
Pan + C	656.90	1086.5				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
26.90	18.8	54.3

Remarks: These results relate to the sample that was tested

GEOTECH SOLUTIONS

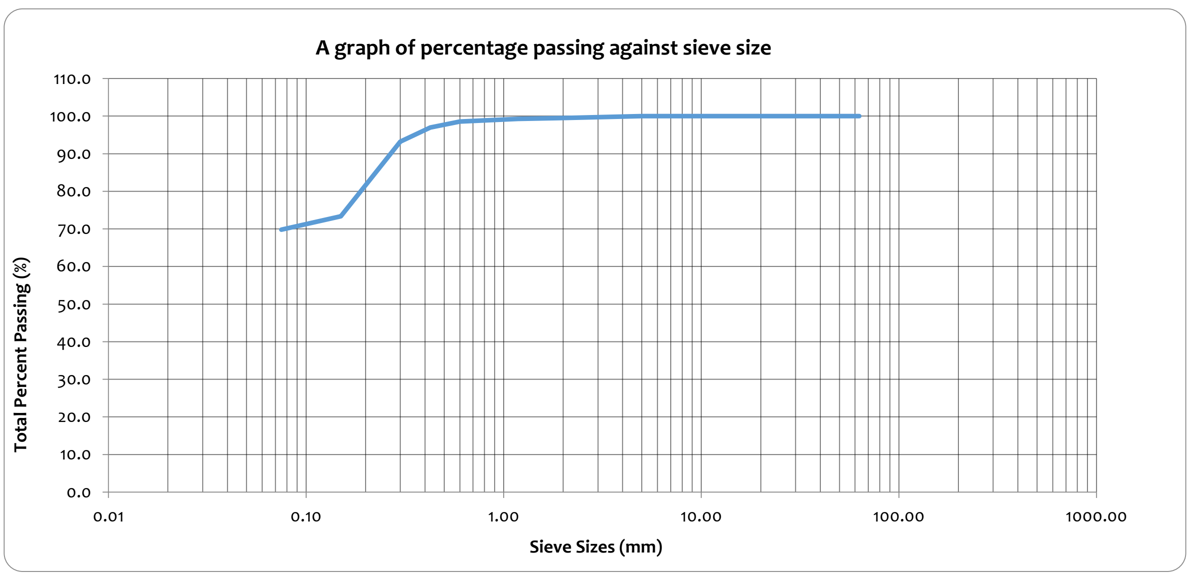
Laboratory Manager

PARTICLE SIZE DISTRIBUTION TEST REPORT

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m)**	9.0 - 10.00
Location:		**Test Method:**	BS 1377: Part 2: 1990
Pit No.:	KL 30 (B103+5)	**Technician:**	Eddy Watema

Dry mass before washing (A)- gm:	1456.30	Dry mass after washing (B)- gm:	342.00	C = A - B (gm)
Mass of dry sample (gm), m_1:	1132.40	Moisture Content (%):	28.60	1114.30

Sieve Sizes (mm)	Mass Retained Mass (g)	Cumulative Retained Mass (g)	% Retained	Cumulative Retained (%)	% Passing	Cumulative % Passing
63.000	0.00	0.0	0.0	0.0	100.0	100.0
50.000	0.00	0.0	0.0	0.0	100.0	100.0
37.500	0.00	0.0	0.0	0.0	100.0	100.0
28.000	0.00	0.0	0.0	0.0	100.0	100.0
20.000	0.00	0.0	0.0	0.0	100.0	100.0
14.000	0.00	0.0	0.0	0.0	100.0	100.0
10.000	0.00	0.0	0.0	0.0	100.0	100.0
6.300	0.00	0.0	0.0	0.0	100.0	100.0
5.000	0.00	0.0	0.0	0.0	100.0	100.0
2.000	5.70	5.7	0.5	0.5	99.5	99.5
1.180	2.50	8.2	0.2	0.7	99.8	99.3
0.600	8.00	16.2	0.7	1.4	99.3	98.6
0.425	18.10	34.3	1.6	3.0	98.4	97.0
0.300	42.40	76.7	3.7	6.8	96.3	93.2
0.150	225.00	301.7	19.9	26.6	80.1	73.4
0.075	40.30	342.0	3.6	30.2	96.4	69.8
Pan	0.00	342.0				
Pan + C	1114.30	1456.3				



GRAVEL (%)	SAND (%)	CLAY & SILT (%)
0.50	29.7	69.8

Remarks: These results relate to the sample that was tested

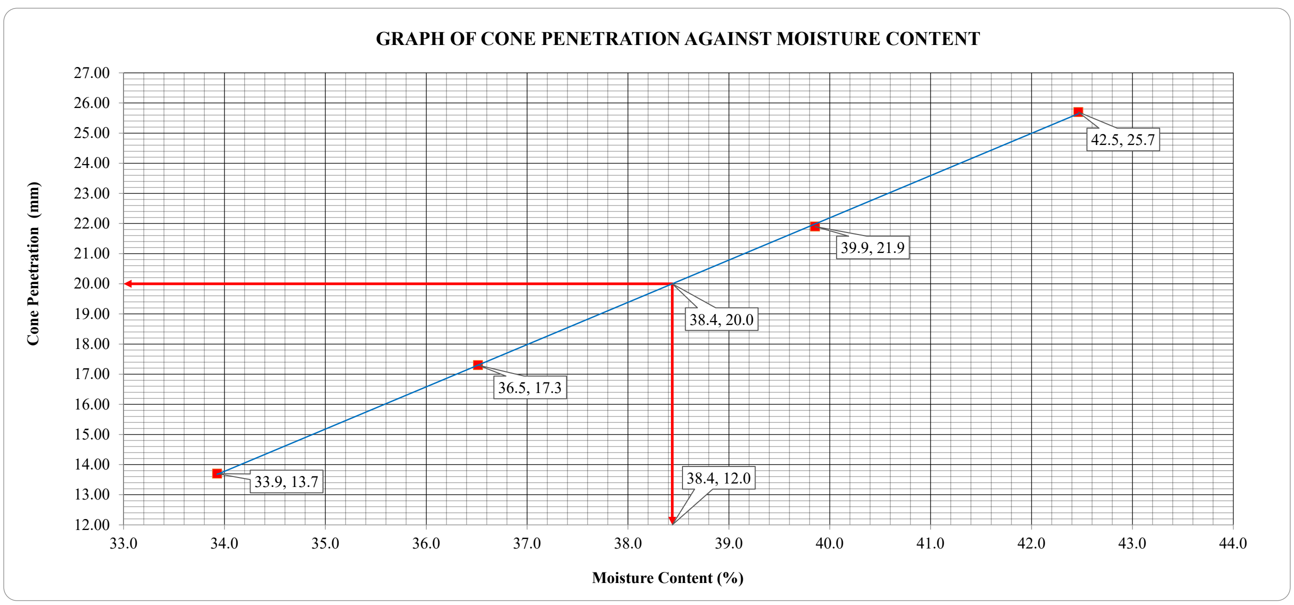
GEOTECH SOLUTIONS

Laboratory Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	0.0 - 0.4
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										PLASTIC LIMIT (%)	
TEST No.		1		2		3		4		1	2
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	13.7	13.7	17.3	17.3	21.9	21.9	25.7	25.7		
Average Penetration	mm	13.70		17.30		21.90		25.70			
Container Number		KZ	KL	IL	CI	CC	IF	ED	RC	DI	WR
Mass of wet soil + container (a):	g	25.92	24.54	21.50	20.06	20.99	25.48	24.77	24.97	22.37	28.57
Mass of dry soil + container (b):	g	22.78	21.44	18.61	18.10	18.54	21.64	20.43	20.57	20.82	25.38
Mass of container (c):	g	13.36	12.46	10.84	12.63	12.50	11.83	10.37	10.04	14.31	11.31
Mass of moisture (d = a - b):	g	3.14	3.10	2.89	1.96	2.45	3.84	4.34	4.40	1.55	3.19
Mass of dry soil (e = b - c):	g	9.42	8.98	7.77	5.47	6.04	9.81	10.06	10.53	6.51	14.07
Moisture content (w = 100 x (d) / (e)):		33.33	34.52	37.19	35.83	40.56	39.14	43.14	41.79	23.81	22.67
Average Moisture Content:		33.9		36.5		39.9		42.5		23.2	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	38.4	**Plasticity Index -PI (%)**	15.2
Plastic Limit -PL (%)	23.2		

Remarks: These results relate to the sample that was tested

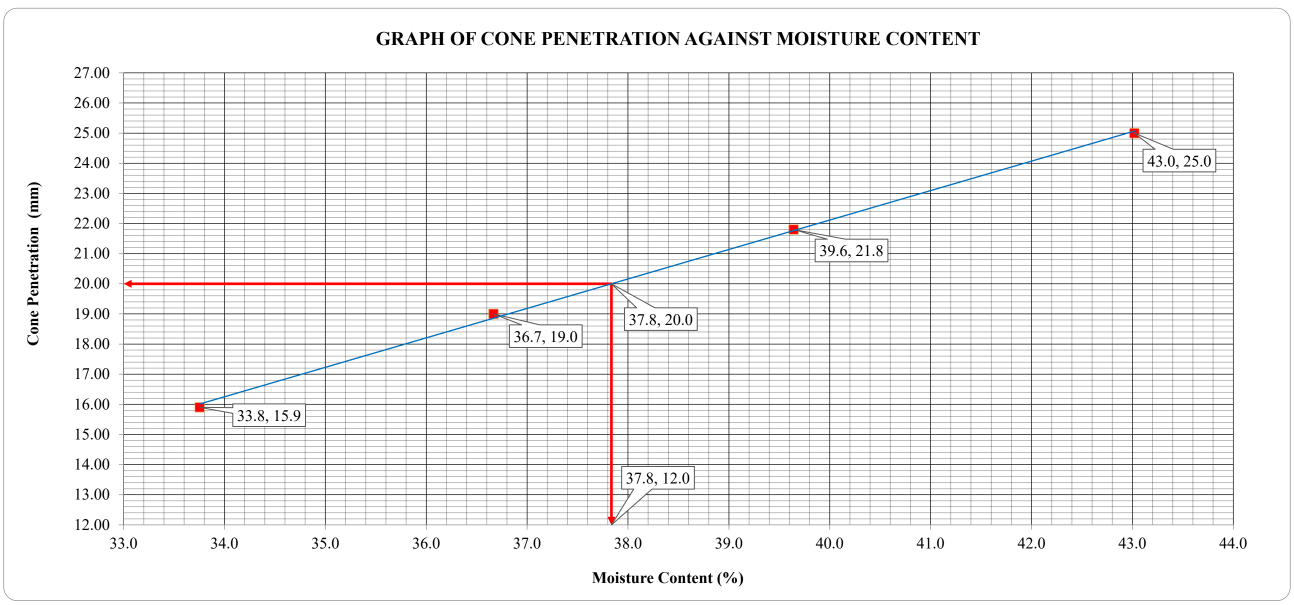
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	0.4 - 1.50
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										PLASTIC LIMIT (%)	
TEST No.		1		2		3		4		1	2
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	15.9	15.9	19.0	19.0	21.8	21.8	25.0	25.0		
Average Penetration	mm	15.90		19.00		21.80		25.00			
Container Number		B5	GZ	8B	N3	4S	6B	8B	4	3B	P5
Mass of wet soil + container (a):	g	30.43	31.30	32.38	33.68	35.08	35.33	38.00	37.00	24.10	23.68
Mass of dry soil + container (b):	g	26.20	26.79	27.33	28.18	28.95	29.10	30.63	29.93	22.15	21.85
Mass of container (c):	g	13.61	13.49	13.53	13.21	13.50	13.37	13.42	13.57	13.41	13.41
Mass of moisture (d = a - b):	g	4.23	4.51	5.05	5.50	6.13	6.23	7.37	7.07	1.95	1.83
Mass of dry soil (e = b - c):	g	12.59	13.30	13.80	14.97	15.45	15.73	17.21	16.36	8.74	8.44
Moisture content (w = 100 x (d) / (e)):		33.60	33.91	36.59	36.74	39.68	39.61	42.82	43.22	22.31	21.68
Average Moisture Content:		33.8		36.7		39.6		43.0		22.0	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = $100(1-L_D/L_o)$**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	37.8	**Plasticity Index -PI (%)**	15.8
Plastic Limit -PL (%)	22.0		

Remarks: These results relate to the sample that was tested

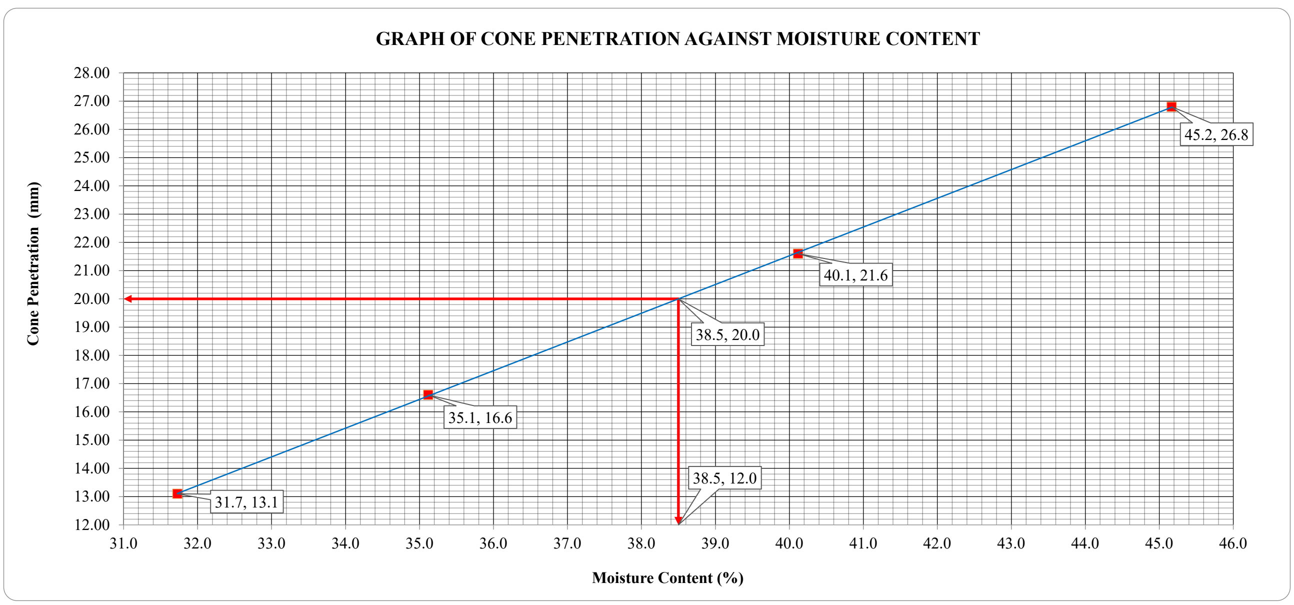
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	1.50 - 2.00
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										PLASTIC LIMIT (%)	
TEST No.		1		2		3		4		1	2
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	13.1	13.1	16.6	16.6	21.6	21.6	26.8	26.8		
Average Penetration	mm	13.10		16.60		21.60		26.80			
Container Number		KP	XF	JZ	SZ	ZU	CM	EP	JP	FS	ZS
Mass of wet soil + container (a):	g	21.03	28.78	21.02	20.13	28.25	28.24	23.43	29.42	26.06	20.24
Mass of dry soil + container (b):	g	19.56	25.30	18.44	18.04	24.35	24.18	20.72	23.48	23.49	19.14
Mass of container (c):	g	14.84	14.53	10.95	12.20	14.48	14.21	14.64	10.50	12.83	14.24
Mass of moisture (d = a - b):	g	1.47	3.48	2.58	2.09	3.90	4.06	2.71	5.94	2.57	1.10
Mass of dry soil (e = b - c):	g	4.72	10.77	7.49	5.84	9.87	9.97	6.08	12.98	10.66	4.90
Moisture content (w = 100 x (d) / (e)):		31.14	32.31	34.45	35.79	39.51	40.72	44.57	45.76	24.11	22.45
Average Moisture Content:		31.7		35.1		40.1		45.2		23.3	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	38.5	**Plasticity Index -PI (%)**	15.2
Plastic Limit -PL (%)	23.3		

Remarks: These results relate to the sample that was tested

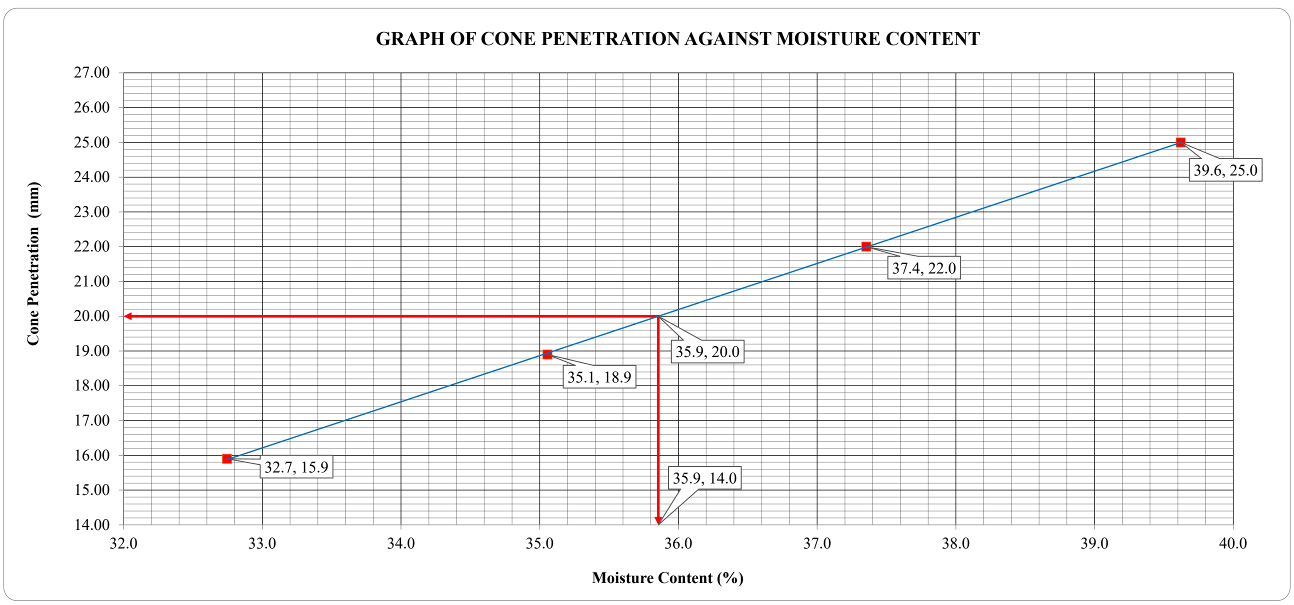
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	2.00 - 3.00
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										**PLASTIC LIMIT (%)**	
TEST No.		**1**		**2**		**3**		**4**		**1**	**2**
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	15.9	15.9	18.9	18.9	22.0	22.0	25.0	25.0		
Average Penetration	**mm**	15.90		18.90		22.00		25.00			
Container Number		**XE**	**4S**	**8K**	**KF**	**P5**	**MV**	**6E**	**8X**	**80**	**P14**
Mass of wet soil + container (a):	**g**	31.14	30.17	33.83	33.36	36.42	35.06	39.21	38.52	24.30	21.55
Mass of dry soil + container (b):	**g**	26.73	26.06	28.51	28.23	30.13	29.19	31.90	31.44	22.80	20.34
Mass of container (c):	**g**	13.24	13.53	13.34	13.59	13.35	13.42	13.42	13.60	13.57	12.93
Mass of moisture (d = a - b):	**g**	4.41	4.11	5.32	5.13	6.29	5.87	7.31	7.08	1.50	1.21
Mass of dry soil (e = b - c):	**g**	13.49	12.53	15.17	14.64	16.78	15.77	18.48	17.84	9.23	7.41
Moisture content (w = 100 x (d) / (e)):		32.69	32.80	35.07	35.04	37.49	37.22	39.56	39.69	16.25	16.33
Average Moisture Content:		32.7		35.1		37.4		39.6		16.3	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	35.9	**Plasticity Index -PI (%)**	19.6
Plastic Limit -PL (%)	16.3		

Remarks: These results relate to the sample that was tested

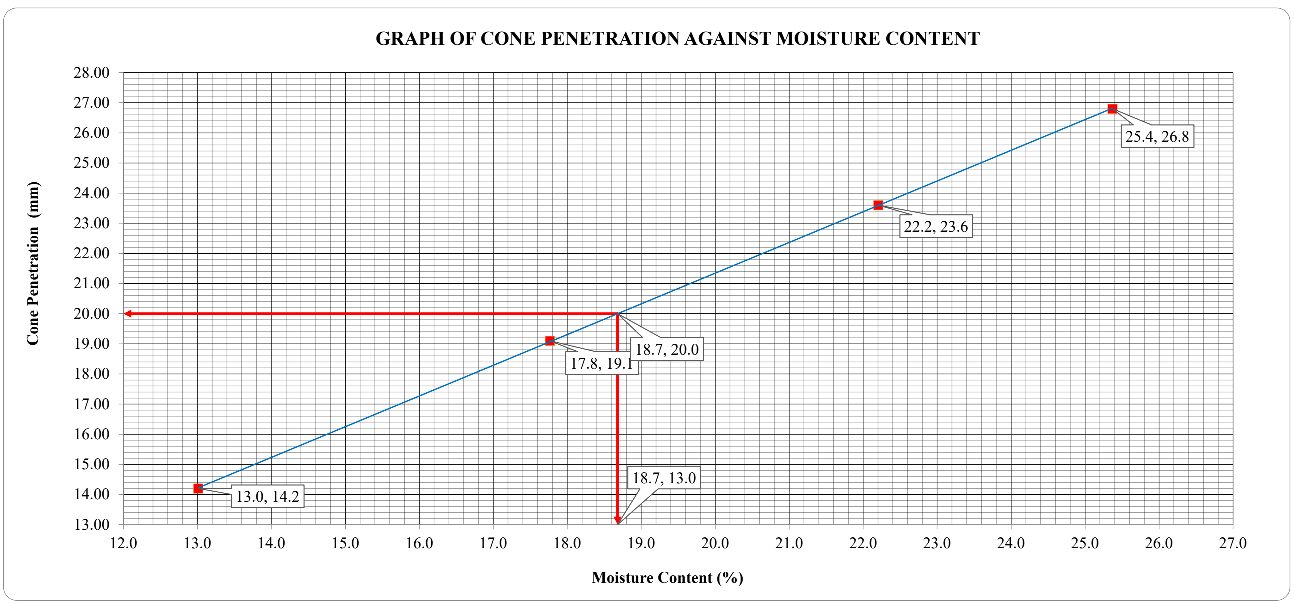
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	3.00 - 4.50
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										PLASTIC LIMIT (%)	
TEST No.		1		2		3		4		1	2
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	14.2	14.2	19.1	19.1	23.6	23.6	26.8	26.8		
Average Penetration	mm	14.20		19.10		23.60		26.80			
Container Number		RS	MT	NU	EM	PK	WV	JP	NF		
Mass of wet soil + container (a):	g	27.09	20.96	28.22	20.42	26.07	26.51	24.56	27.11		
Mass of dry soil + container (b):	g	25.07	20.29	25.58	19.20	24.00	23.55	22.55	24.04		
Mass of container (c):	g	10.26	14.88	10.48	12.44	14.34	10.67	14.85	11.58		
Mass of moisture (d = a - b):	g	2.02	0.67	2.64	1.22	2.07	2.96	2.01	3.07	0.00	0.00
Mass of dry soil (e = b - c):	g	14.81	5.41	15.10	6.76	9.66	12.88	7.70	12.46	0.00	0.00
Moisture content (w = 100 x (d) / (e)):		13.64	12.38	17.48	18.05	21.43	22.98	26.10	24.64	#DIV/0!	#DIV/0!
Average Moisture Content:		13.0		17.8		22.2		25.4		#DIV/0!	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT

Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX

Liquid Limit -LL (%)	18.7	**Plasticity Index -PI (%)**	#DIV/0!
Plastic Limit -PL (%)	#DIV/0!		

Remarks: These results relate to the sample that was tested

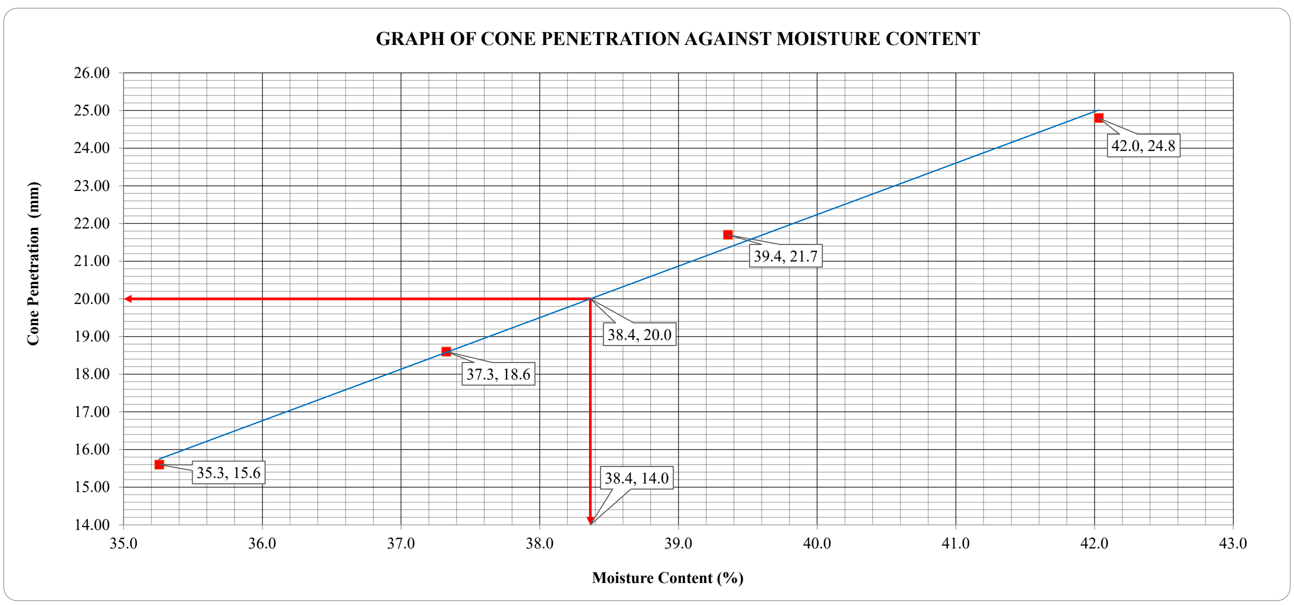
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	4.50 - 5.10
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										PLASTIC LIMIT (%)	
TEST No.		**1**		**2**		**3**		**4**		**1**	**2**
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	15.6	15.6	18.6	18.6	21.7	21.7	24.8	24.8		
Average Penetration	**mm**	15.60		18.60		21.70		24.80			
Container Number		**S4**	**2R**	**1B**	**V3**	**8Z**	**GB**	**18C**	**3B**	**LA**	**YX**
Mass of wet soil + container (a):	**g**	30.48	30.13	32.99	32.55	34.40	34.42	39.59	39.14	24.47	24.60
Mass of dry soil + container (b):	**g**	25.98	25.65	27.66	27.35	28.46	28.38	31.83	31.50	22.90	22.98
Mass of container (c):	**g**	13.24	12.92	13.40	13.40	13.40	13.00	13.38	13.31	13.44	13.22
Mass of moisture (d = a - b):	**g**	4.50	4.48	5.33	5.20	5.94	6.04	7.76	7.64	1.57	1.62
Mass of dry soil (e = b - c):	**g**	12.74	12.73	14.26	13.95	15.06	15.38	18.45	18.19	9.46	9.76
Moisture content (w = 100 x (d) / (e)):		35.32	35.19	37.38	37.28	39.44	39.27	42.06	42.00	16.60	16.60
Average Moisture Content:		35.3		37.3		39.4		42.0		16.6	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = $100(1-L_D/L_o)$**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	38.4	**Plasticity Index -PI (%)**	21.8
Plastic Limit -PL (%)	16.6		

Remarks: These results relate to the sample that was tested

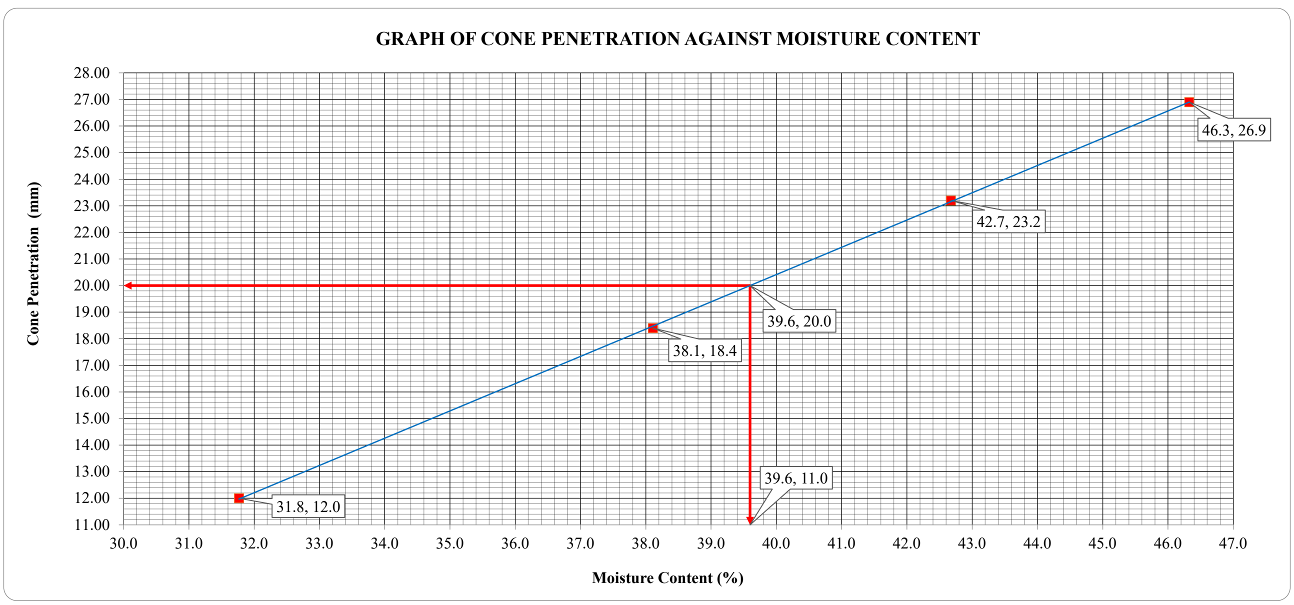
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	5.10 - 6.00
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										PLASTIC LIMIT (%)	
TEST No.		1		2		3		4		1	2
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	12.0	12.0	18.4	18.4	23.2	23.2	26.9	26.9		
Average Penetration	mm	12.00		18.40		23.20		26.90			
Container Number		HH	QZ	WX	MQ	ML	TM	NQ	LL	YE	EH
Mass of wet soil + container (a):	g	24.44	20.94	21.92	20.25	21.26	28.90	22.09	26.05	25.93	20.79
Mass of dry soil + container (b):	g	21.33	19.10	19.19	18.75	18.58	23.95	19.83	22.19	23.51	19.37
Mass of container (c):	g	11.70	13.21	11.94	14.86	12.41	12.14	14.89	13.96	10.97	11.49
Mass of moisture (d = a - b):	g	3.11	1.84	2.73	1.50	2.68	4.95	2.26	3.86	2.42	1.42
Mass of dry soil (e = b - c):	g	9.63	5.89	7.25	3.89	6.17	11.81	4.94	8.23	12.54	7.88
Moisture content (w = 100 x (d) / (e)):		32.29	31.24	37.66	38.56	43.44	41.91	45.75	46.90	19.30	18.02
Average Moisture Content:		31.8		38.1		42.7		46.3		18.7	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	39.6	**Plasticity Index -PI (%)**	20.9
Plastic Limit -PL (%)	18.7		

Remarks: These results relate to the sample that was tested

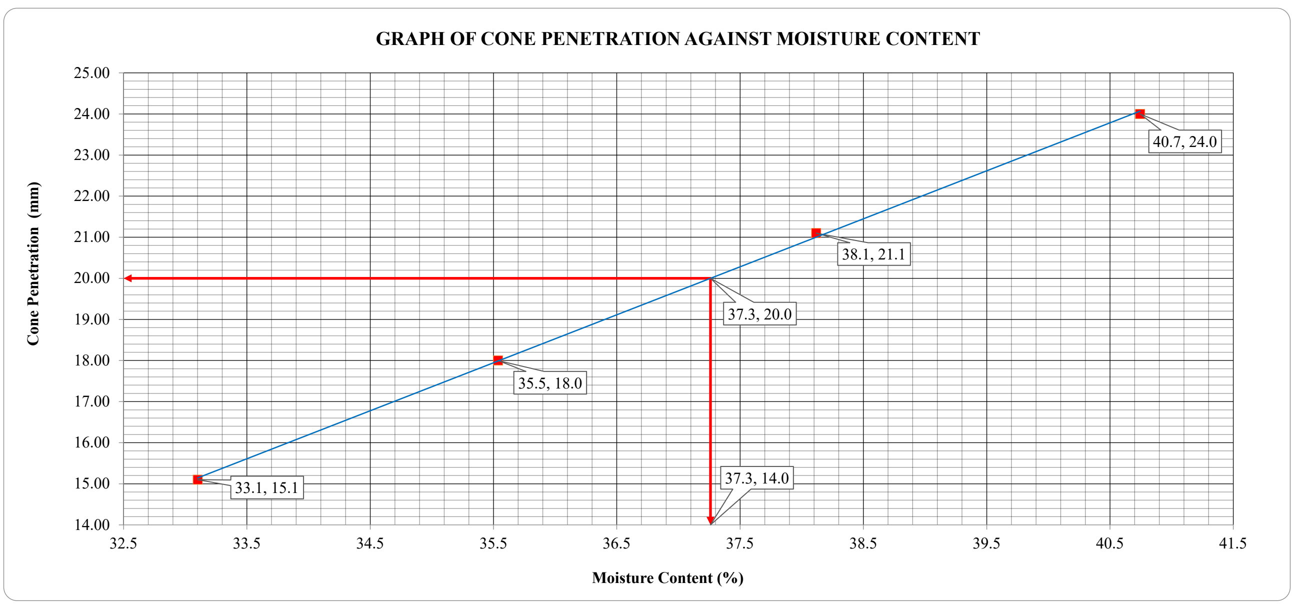
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	6.00 - 7.00
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										**PLASTIC LIMIT (%)**	
TEST No.		**1**		**2**		**3**		**4**		**1**	**2**
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	15.1	15.1	18.0	18.0	21.1	21.1	24.0	24.0		
Average Penetration	**mm**	15.10		18.00		21.10		24.00			
Container Number		**N6**	**GA**	**B6**	**4B**	**8S**	**BK**	**B8**	**B57**	**KL**	**GX**
Mass of wet soil + container (a):	**g**	30.64	30.60	32.24	32.03	35.01	34.94	39.03	38.20	25.88	25.47
Mass of dry soil + container (b):	**g**	26.34	26.35	27.24	27.20	29.03	29.02	31.61	31.09	23.99	23.65
Mass of container (c):	**g**	13.36	13.50	13.25	13.53	13.39	13.44	13.43	13.61	13.33	13.48
Mass of moisture (d = a - b):	**g**	4.30	4.25	5.00	4.83	5.98	5.92	7.42	7.11	1.89	1.82
Mass of dry soil (e = b - c):	**g**	12.98	12.85	13.99	13.67	15.64	15.58	18.18	17.48	10.66	10.17
Moisture content (w = 100 x (d) / (e)):		33.13	33.07	35.74	35.33	38.24	38.00	40.81	40.68	17.73	17.90
Average Moisture Content:		33.1		35.5		38.1		40.7		17.8	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		
PLASTICITY INDEX			
Liquid Limit -LL (%)	37.3	**Plasticity Index -PI (%)**	19.4
Plastic Limit -PL (%)	17.8		

Remarks: These results relate to the sample that was tested

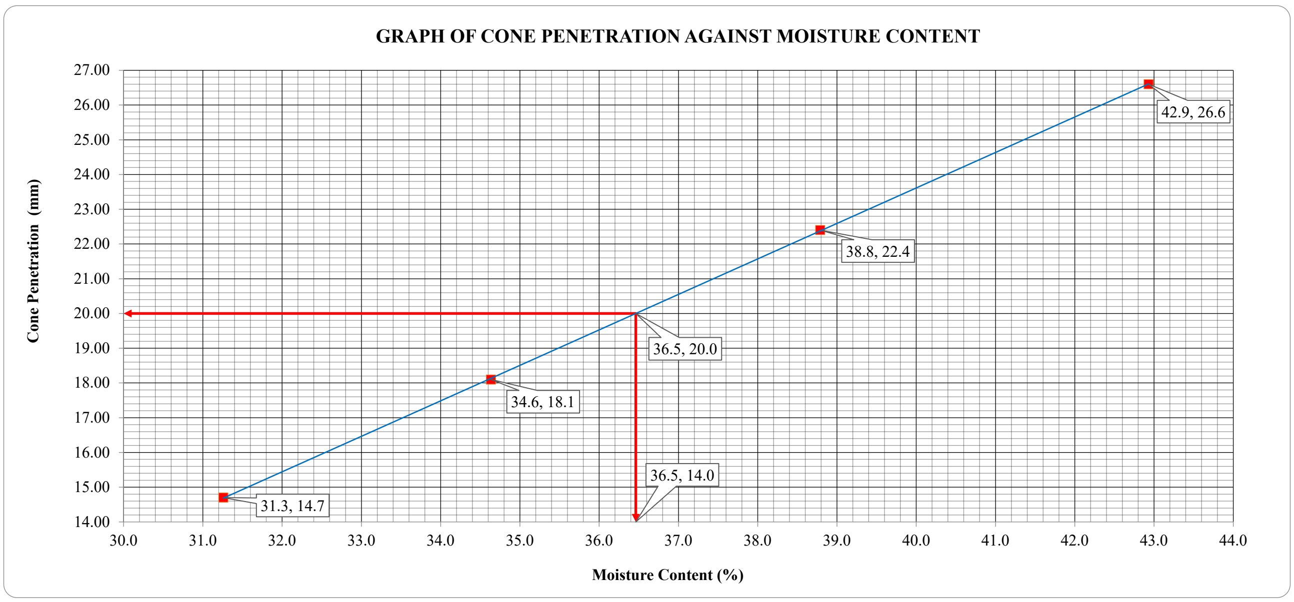
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	7.00 - 7.80
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										**PLASTIC LIMIT (%)**	
TEST No.		**1**		**2**		**3**		**4**		**1**	**2**
Initial dial gauge reading	**mm**	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	**mm**	14.7	14.7	18.1	18.1	22.4	22.4	26.6	26.6		
Average Penetration	**mm**	14.70		18.10		22.40		26.60			
Container Number		**MR**	**UA**	**TA**	**JO**	**VZ**	**MK**	**SX**	**LM**	**HD**	**NN**
Mass of wet soil + container (a):	**g**	29.60	29.22	28.87	21.17	21.39	24.64	20.90	25.87	22.21	26.69
Mass of dry soil + container (b):	**g**	25.05	25.68	24.63	19.03	19.42	20.75	18.91	21.74	20.93	24.63
Mass of container (c):	**g**	10.15	14.61	12.10	12.99	14.42	10.56	14.20	12.27	13.46	11.71
Mass of moisture (d = a - b):	**g**	4.55	3.54	4.24	2.14	1.97	3.89	1.99	4.13	1.28	2.06
Mass of dry soil (e = b - c):	**g**	14.90	11.07	12.53	6.04	5.00	10.19	4.71	9.47	7.47	12.92
Moisture content (w = 100 x (d) / (e)):		30.54	31.98	33.84	35.43	39.40	38.17	42.25	43.61	17.14	15.94
Average Moisture Content:		31.3		34.6		38.8		42.9		16.5	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	36.5	**Plasticity Index -PI (%)**	19.9
Plastic Limit -PL (%)	16.5		

Remarks: These results relate to the sample that was tested

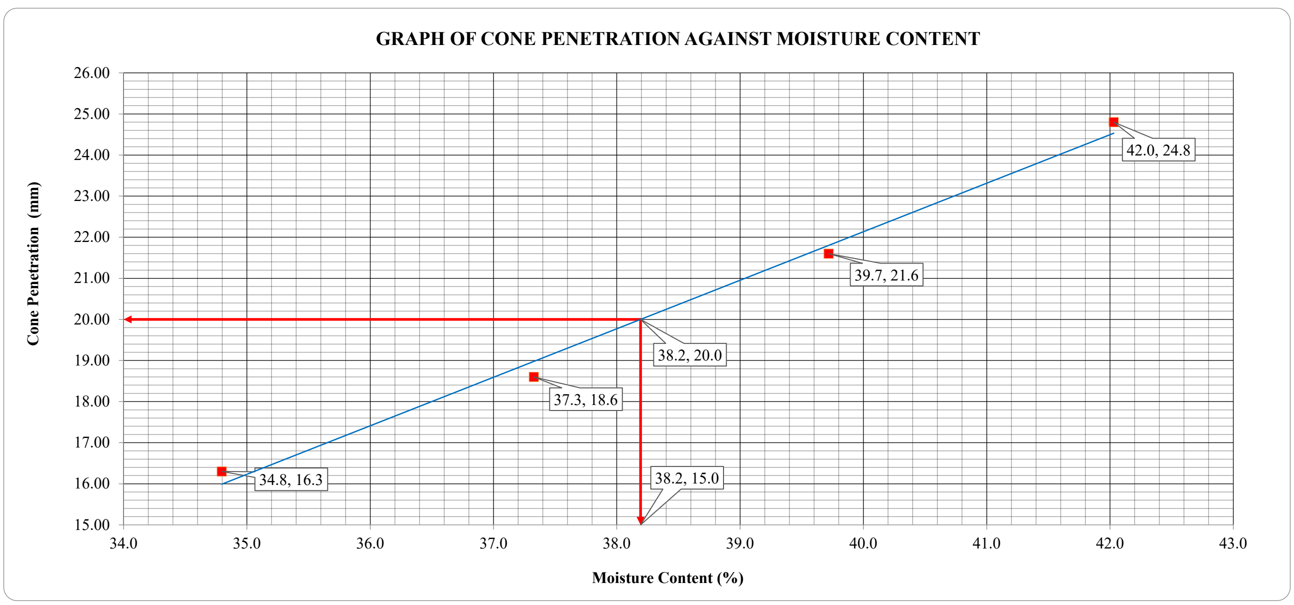
GEOTECH SOLUTIONS

Technical Manager

DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	7.80 - 9.00
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD										**PLASTIC LIMIT (%)**	
TEST No.		1		2		3		4		1	2
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0		
Final dial gauge reading	mm	16.3	16.3	18.6	18.6	21.6	21.6	24.8	24.8		
Average Penetration	mm	16.30		18.60		21.60		24.80			
Container Number		6E	XA	GA	M2	BC	SB	3B	10M	PG	B9
Mass of wet soil + container (a):	g	30.43	30.13	32.99	32.55	34.49	34.44	39.59	39.14	24.49	24.63
Mass of dry soil + container (b):	g	25.98	25.70	27.66	27.35	28.48	28.36	31.83	31.50	22.90	22.95
Mass of container (c):	g	13.24	12.92	13.40	13.40	13.40	13.00	13.38	13.31	13.44	13.22
Mass of moisture (d = a - b):	g	4.45	4.43	5.33	5.20	6.01	6.08	7.76	7.64	1.59	1.68
Mass of dry soil (e = b - c):	g	12.74	12.78	14.26	13.95	15.08	15.36	18.45	18.19	9.46	9.73
Moisture content (w = 100 x (d) / (e)):		34.93	34.66	37.38	37.28	39.85	39.58	42.06	42.00	16.81	17.27
Average Moisture Content:		34.8		37.3		39.7		42.0		17.0	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	38.2	**Plasticity Index -PI (%)**	21.2
Plastic Limit -PL (%)	17.0		

Remarks: These results relate to the sample that was tested

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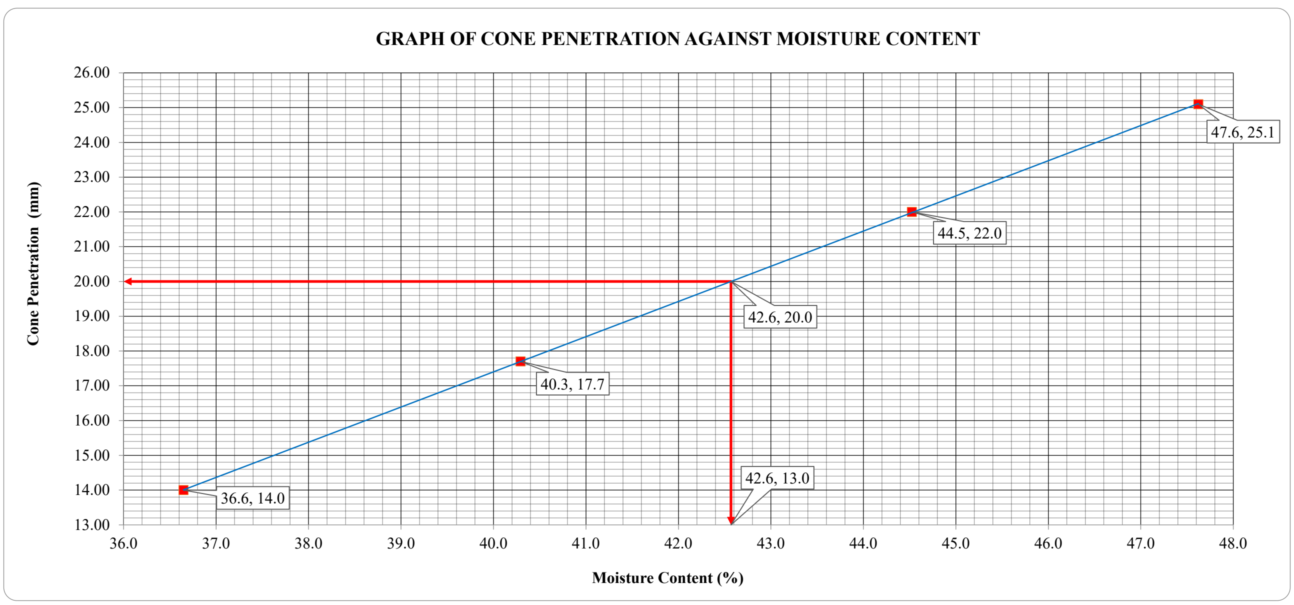
DETERMINATION OF ATTERBERG LIMITS (BS 1377-2: 1990)

Project:	GEOTECHNICAL INVESTIGATIONS ALONG THE 132 KV KARUMA-LIRA TRANSMISSION LINE		
Client:	SAMUEL ACIDRI	**Depth (m):**	9.00 - 10.00
Location:	LIRA LINE	**Testing date:**	
Sampling Date:		**Test method:**	BS 1377: PART 2: 1990
Bore Hole No.:	KL 30 (B103+5)	**Technician:**	Nambi Martha
Job ref. No.:		**Checked By:**	Godfrey Lubanga
Soil Description:			

LIQUID LIMIT DETERMINATION BY CONE PENETRATION METHOD											PLASTIC LIMIT (%)	
TEST No.		1		2		3		4			1	2
Initial dial gauge reading	mm	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0			
Final dial gauge reading	mm	14.0	14.0	17.7	17.7	22.0	22.0	25.1	25.1			
Average Penetration	mm	14.00		17.70		22.00		25.10				
Container Number		UX	QZ	CS	OW	DD	IW	HH	US		XU	AK
Mass of wet soil + container (a):	g	26.27	28.59	23.93	21.94	27.35	23.15	29.46	29.53		22.26	28.86
Mass of dry soil + container (b):	g	23.09	23.68	19.99	18.49	22.92	20.02	24.72	24.34		20.17	25.40
Mass of container (c):	g	14.57	10.03	10.08	10.04	12.81	13.10	14.91	13.28		13.97	14.69
Mass of moisture (d = a - b):	g	3.18	4.91	3.94	3.45	4.43	3.13	4.74	5.19		2.09	3.46
Mass of dry soil (e = b - c):	g	8.52	13.65	9.91	8.45	10.11	6.92	9.81	11.06		6.20	10.71
Moisture content (w = 100 x (d) / (e)):		37.32	35.97	39.76	40.83	43.82	45.23	48.32	46.93		33.71	32.31
Average Moisture Content:		36.6		40.3		44.5		47.6			33.0	



LINEAR SHRINKAGE & SHRINKAGE PRODUCT			
Oven dried Length (L_D) (mm)	122.00	**Linear Shrinkage LS = 100(1-L_D/L_o)**	12.86
Initial Length (L_O) (mm)	140.00		

PLASTICITY INDEX			
Liquid Limit -LL (%)	42.6	**Plasticity Index -PI (%)**	9.6
Plastic Limit -PL (%)	33.0		

Remarks: These results relate to the sample that was tested

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Appendix B.7 - Chemical Analysis Tests

Table B.7-1: Summary for Water Chemical Test Results for KL 30 (B103+5)

WATER CHEMICAL RESULTS (BS 1377: Part 3: 1990)					
Tower Location	**Depth (m)**	**Chloride content (g/L)**	**Sulphate content (g/L)**	**pH**	**Group Symbol**
KL 30 (B103+5)	1.140	10	0.0686	6.27	CI

Table B.7-2: Soil Chemical Test Results for AP 108/15

CHEMICAL RESULTS FOR AP 108/15								
Tower Location	**Depth (m)**	**Specific Gravity**	**Sulphate Content**	**Chloride Content**	**Bulk Density**	**pH**	**USCS Classification**	**Group Symbol**
AP 108/15	1.00	2.777	0.06	0.007	1.820	5.27	Clayey SAND	SC
	2.45	2.380	0.06	0.007	1.803	5.61	Clayey SAND	SC
	4.20	2.370	0.05	0.007	1.803	5.38	Silty SAND	SM

Table B.7-3: Soil Chemical Test Results for AP 108/20

CHEMICAL RESULTS FOR AP 108/20								
Tower Location	**Depth (m)**	**Specific Gravity**	**Sulphate Content**	**Chloride Content**	**Bulk Density**	**pH**	**USCS Classification**	**Group Symbol**
AP 108/20	0.0-1.0	2.483	0.09	0.008	1.678	5.06	Silty SAND with Gravel	SM
	1.0-2.0	2.380	0.06	0.009	1.633	5.14	Silty SAND with Gravel	SM
	2.0-3.0	2.412	0.06	0.009	1.696	5.22	Clayey SAND with Gravel	SC

Table B.7-4: Chemical Analysis Results Summary for AP 104/5

CHEMICAL ANALYSIS RESULTS SUMMARY FOR AP 104/5								
Tower Location	**Depth (m)**	**Specific Gravity**	**pH**	**Sulphate content**	**Chloride content**	**Bulk density**	**USCS Soil Classification**	**Group Symbol**
AP 104/5	0.0-1.0	2.65	6.00	Absent	0.026	1.712	Sandy Lean CLAY	CL
	1.0-3.5	2.66	7.10	Absent	0.040	1.725	Sandy Lean CLAY	CL
	3.5-5.0	2.71	6.68	Absent	0.038	1.795	Clayey SAND	SC
	5.0-8.0	2.70	6.85	Traces	0.030	1.790	Clayey SAND	SC
	8.0-10.0	2.64	5.90	Absent	0.015	1.710	Sandy Lean CLAY	CL
	10.0-12.0	2.66	6.01	Absent	0.052	1.705	Sandy Lean CLAY	CL
	12.0-15.0	2.71	6.10	Absent	0.021	1.772	Clayey SAND	SC
	15.0-16.3	2.69	5.88	Absent	0.015	1.768	Clayey SAND	SC
	16.3-18.5	2.72	5.94	Absent	0.057	1.789	Silty SAND	SM
	18.5-20.0	2.73	5.96	Absent	0.024	1.782	Silty SAND	SM

Appendix B.8 - Shear and Consolidation Test Results

Table B.8-1: Shear Test Bearing Capacity Results for AP 104/5

GEOTECH SOLUTIONS	**SHEAR TEST BEARING CAPACITY TEST REPORT FOR AP 104/5**		
Project:	Geotechnical Investigation for AP 104/5 Karuma-Kawanda Transmission Line (400 kV)		
Client:	Samuel Acidri		
Location:	AP 104/5 (400 kV Karuma-Kawanda TL)		
Sampling Date:	N/A	**Testing Date:**	N/A
Technician:	EO	**Checked by:**	BK

EVALUATION OF BEARING CAPACITIES BASED ON GENERAL SHEAR FAILURE

Location	**Depth**	**Width (B)**	**Bulk Density**	**Cohesion (c)**	**Friction angle (ϕ)**	**Bearing capacity factors**			**Ultimate bearing capacity (q_{ult})**	**Factor of Safety**	**Allowable bearing capacity (q_{all})**
	(m)	**(m)**	**(Mg/m^3)**	**(kPa)**	**(°)**	**Nc**	**Nq**	**Ny**	**(kPa)**	**(FoS)**	**(kPa)**
AP 104/5	5.20	1	1.790	12.30	21	18.92	8.26	4.31	1025	3	342
	10.40	1	1.745	18.50	15	12.86	4.45	1.52	1043	3	348

Remarks:

1) The General Shear Failure was considered in design along with a strip foundation at the respective depth of B = 1.0m.
2) Terzaghi's Formula for Ultimate Soil Bearing Capacity of a Square Footing was used as shown below:

$$q_{all} = \frac{q_{ult}}{FOS} = cN_c + 0.5\gamma BN_\gamma + qN_q$$

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Table B.8-2: Shaft resistance and end bearing resistance for AP 104/5

Borehole Location	Depth (m)	Predominate soil particles	Average N_{55}	Rod length correction, η_2	Unit weight (kN/m^3)	Effective Unit Weight (kN/m^3)	Average $(N_1)_{60}$	Adhesion Factor, (α)	Unconfined compressive strength, (q_u)	Undrained shear strength, (S_u)	Effective angle of internal friction, (ϕ')	N_q	Effective overburden pressure, $(\bar{q})$	Shaft resistance, $f_s\,(kPa)$	End Bearing resistance, $q_b\,(KPa)$
AP 104/5	1.00	Clay	2	0.75	18.00	8.19	-	0.45	24	12	-	-	11.1	5.4	108.0
	2.30	Clay	4	0.75	18.00	8.19	-	0.45	48	24	-	-	21.8	10.8	216.0
	3.60	Clay	7	0.75	18.00	8.19	-	0.45	84	42	-	-	32.4	18.9	378.0
	4.90	Sand	15	0.85	18.50	8.69	16	0.55	180	-	34	30	43.7	15.4	1311.7
	6.20	Sand	19	0.95	18.50	8.69	18	0.55	228	-	36	30	55.0	423.9	1650.6
	7.50	Sand	19	0.95	18.50	8.69	18	0.55	228	-	36	30	66.3	510.8	1989.5
	8.80	Sand	19	0.95	18.50	8.69	18	0.55	228	-	36	30	77.6	597.6	2328.5
	10.10	Sand	32	1.00	18.50	8.69	29	0.55	384	-	36	60	88.9	684.4	5334.7
	11.40	Sand	31	1.00	18.50	8.69	28	0.55	372	-	36	60	100.2	771.3	6012.5
	12.70	Sand	21	1.00	18.50	8.69	19	0.55	252	-	36	30	111.5	858.1	3345.2
	14.00	Sand	21	1.00	18.50	8.69	19	0.55	252	-	36	30	122.8	945.0	3684.1
	15.30	Sand	26	1.00	18.50	8.69	24	0.55	312	-	36	30	134.1	1031.8	4023.0
	16.60	Sand	24	1.00	18.50	8.69	22	0.55	288	-	36	30	145.4	1118.6	4361.9
	17.90	Sand	25	1.00	18.50	8.69	23	0.55	300	-	36	30	156.7	1205.5	4700.8
	19.20	Sand	27	1.00	18.50	8.69	25	0.55	324	-	36	60	168.0	1292.3	10000.0

If the foundation is constructed with drilling fluids and there is uncertainty on the base conditions, then design is based on no or reduced load carrying capacity on the base. If the movement required to mobilize the base is unacceptable then no base Bearing Capacity is used.

Table B.8-3: The shear test results for AP 104/5

Location	Depth	Width	Width Bulk Density	Cohesion (c)	Friction angle (ϕ)
	(m)	(m)	(Mg/m^3)	(kPa)	(°)
AP 104/5	5.20	1	1.790	12.30	21
	10.40	1	1.745	18.50	15

Table B.8-4: Consolidation Test Results for AP 104/5

Location	Test Depth	Initial void ratio (e_o)	Initial bulk density (γ_b)	Coefficient of consolidation (c_v)	Coefficient of volume compressibility, (m_v)	Pre-consolidation pressure, (p_o)
	(m)	(-)	(Mg/m^3)	(cm^2/sec)	(m^2/MN)	(kPa)
AP 104/5	10.4-10.7	0.752	1.745	0.0042	0.187	201

Table B.8-5: Bearing Capacity comparison of Shear Strength Test and SPT

Location	Depth	Shear Strength Allowable Bearing Capacity, (q_{all})	SPT Allowable Bearing Capacity, (q_{all})
	(m)	(kPa)	(kPa)
AP 104/5	5.20	342	372.4
	10.40	348	798.0

Table B.8-6: Overburden Pressure Results for KL 30 (B103+5)

<table>
<tr><td rowspan="3">Geotechnical Investigations</td><td colspan="4">Geotechnical Soil Laboratory:</td><td colspan="3">Location Coordinates</td></tr>
<tr><td colspan="2">Profiled by: J.R. Odeke</td><td colspan="2">Trial Pit No: KL 30</td><td>Easting</td><td colspan="2">Northing</td></tr>
<tr><td colspan="2">Date: N/A</td><td colspan="2">KL 30 (B103+5)</td><td>426605.777</td><td colspan="2">246178.091</td></tr>
</table>

<table>
<tr><th rowspan="2">Tower Location</th><th colspan="3">Depth (m)</th><th>Bulk Density</th><th>Vertical Stress per layer</th><th>Overburden Pressure</th><th>Effective Overburden Pressure ($\overline{q}$)</th></tr>
<tr><th>From</th><th>to</th><th>Interval</th><th>(g/cm^3)</th><th>(kPa)</th><th>(kPa)</th><th>(kPa)</th></tr>
<tr><td rowspan="11">KL 30 (B103+5)</td><td>0.00</td><td>0.40</td><td>0.40</td><td>1.681</td><td>6.6</td><td>6.6</td><td>6.6</td></tr>
<tr><td>0.40</td><td>1.00</td><td>0.60</td><td>1.685</td><td>9.9</td><td>16.5</td><td>16.5</td></tr>
<tr><td>1.00</td><td>1.50</td><td>0.50</td><td>1.685</td><td>8.3</td><td>24.8</td><td>24.8</td></tr>
<tr><td>1.50</td><td>2.00</td><td>0.50</td><td>1.741</td><td>8.5</td><td>33.3</td><td>33.3</td></tr>
<tr><td>2.00</td><td>3.00</td><td>1.00</td><td>1.745</td><td>17.1</td><td>50.4</td><td>50.4</td></tr>
<tr><td>3.00</td><td>4.00</td><td>1.00</td><td>2.241</td><td>22.0</td><td>72.4</td><td>72.4</td></tr>
<tr><td>4.00</td><td>4.50</td><td>0.50</td><td>2.241</td><td>11.0</td><td>83.4</td><td>83.4</td></tr>
<tr><td>4.50</td><td>5.10</td><td>0.60</td><td>1.763</td><td>10.4</td><td>93.8</td><td>93.8</td></tr>
<tr><td>5.10</td><td>6.00</td><td>0.90</td><td>1.765</td><td>15.6</td><td>109.4</td><td>109.4</td></tr>
<tr><td>6.00</td><td>7.00</td><td>1.00</td><td>1.785</td><td>17.5</td><td>126.9</td><td>126.9</td></tr>
<tr><td>7.00</td><td>7.50</td><td>0.50</td><td>1.754</td><td>8.6</td><td>135.5</td><td>135.5</td></tr>
<tr><td rowspan="3">KL 30 (B103+5)</td><td>7.50</td><td>7.80</td><td>0.30</td><td>1.754</td><td>5.2</td><td>140.6</td><td>140.6</td></tr>
<tr><td>7.80</td><td>9.00</td><td>1.20</td><td>1.763</td><td>20.8</td><td>161.4</td><td>161.4</td></tr>
<tr><td>9.00</td><td>10.00</td><td>1.00</td><td>1.745</td><td>17.1</td><td>178.5</td><td>178.5</td></tr>
</table>

Table B.8-7: Design Properties of Soils and Concrete as per Technical Specification Schedule 4

S/No.	Normal Foundations	Units	Good Soil	Poor Soil	Soft Rock	Hard Rock	Waterlogged Ground
1	Assumed mass of earth for foundations	kg/m^2	1600	1450	1600	1600	1000
2	Assumed mass of rock for foundations	kg/m^2	-	-	1900	2000	-
3	Assumed mass of concrete for foundations	kg/m^2	2300	2300	2300	2300	1300
4	Assumed ultimate bearing capacity for foundations under specified maximum ultimate loading, including factor of safety	t/m^2	30	15	60	100	15
5	Ultimate shear stress in rock	t/m^2	-	-	3.0	7.5	-
6	Assumed angle of vertical of frustrum of earth resisting uplift (angle of repose)	-	30°	15°	-	-	15°
7	Ultimate plain concrete bearing stress	kg/cm^2	60	60	60	60	60
8	Ultimate adhesion value between galvanised steel and concrete, including factor of safety	kg/cm^2	10.0	10.0	10.0	10.0	10.0
9	Minimum portion of stub loads to be considered in the design of cleats	-	50%	50%	50%	100%	50%

The design considerations to be adopted for the foundations are as indicated in the table above as per the Technical Specification in design for properties of soil and concrete.

Appendix C.1 - Concrete Cube Test Results

Table C.1-1: Compressive Concrete Cube Crushing Strength Test Results

COMPRESSIVE CONCRETE CUBE CRUSHING RESULTS											
Location	**Casting Date**	**No. of Days**	**Testing Date**	**Cube No.**	**Weight (kg)**	**Force (kN)**	**Area m^2**	**Strength (MPa)**	**Average (MPa)**	**Design Strength (Mpa)**	**Remarks**
KL 30 (B103+5)	09/02/2019	7 Days	16/02/2019	1	8400	593.3	0.0225	26.37	27.71	25	Passed
				2	8364	631.7	0.0225	28.08			
				3	8277	645.3	0.0225	28.68			
		28 Days	09/03/2019	1	8293	906.2	0.0225	40.28	40.08		Passed
				2	8545	999.4	0.0225	44.42			
				3	8277	799.5	0.0225	35.53			
AP 108/15	12/03/2019	7 Days	19/03/2019	1	8344	711	0.0225	31.60	29.81	25	Passed
				2	8352	716.3	0.0225	31.84			
				3	8347	584.6	0.0225	25.98			
		28 Days	09/04/2019	1	8452	973.6	0.0225	43.27	43.31		Passed
				2	8432	962	0.0225	42.76			
				3	8443	987.6	0.0225	43.89			
AP 108/20	18/03/2019	7 Days	25/03/2019	1	8220	526.8	0.0225	23.41	26.05	25	Passed
				2	8255	639.5	0.0225	28.42			
				3	8126	591.9	0.0225	26.31			
		28 Days	15/04/2019	1	8313	867.4	0.0225	38.55	38.78		Passed
				2	8330	821.9	0.0225	36.53			
				3	8295	928.1	0.0225	41.25			
AP 104/5	02/05/2019	7 Days	09/05/2019	1	8100	650.8	0.0225	28.92	30.84	25	Passed
				2	8200	686.7	0.0225	30.52			
				3	8250	744.4	0.0225	33.08			
		28 Days	30/05/2019	1	8350	1203.4	0.0225	53.48	52.77		Passed
				2	8450	1235.7	0.0225	54.92			
				3	8400	1122.7	0.0225	49.90			

Appendix C.2 - Density Test Results

Table C.2-1: Compaction Test Summary for KL 30 (B103+5)

Project:	132 kV Karuma - Lira Transmission Line in Uganda.
Client:	Samuel Acidri/Sinohydro Corp.

COMPACTION TEST: Maximum Dry Density (MDD) & Optimum Moisture Content (OMC)
(BS 1377: Part 4: 1990)

Location:	KL 30	Depth of sample:	3.00m - 4.50m
Date of sampling:	1/03/2019	Material Description:	Reddish-brown sandy clays
Date of testing:	8/03/2019	Tested by:	Justus W.

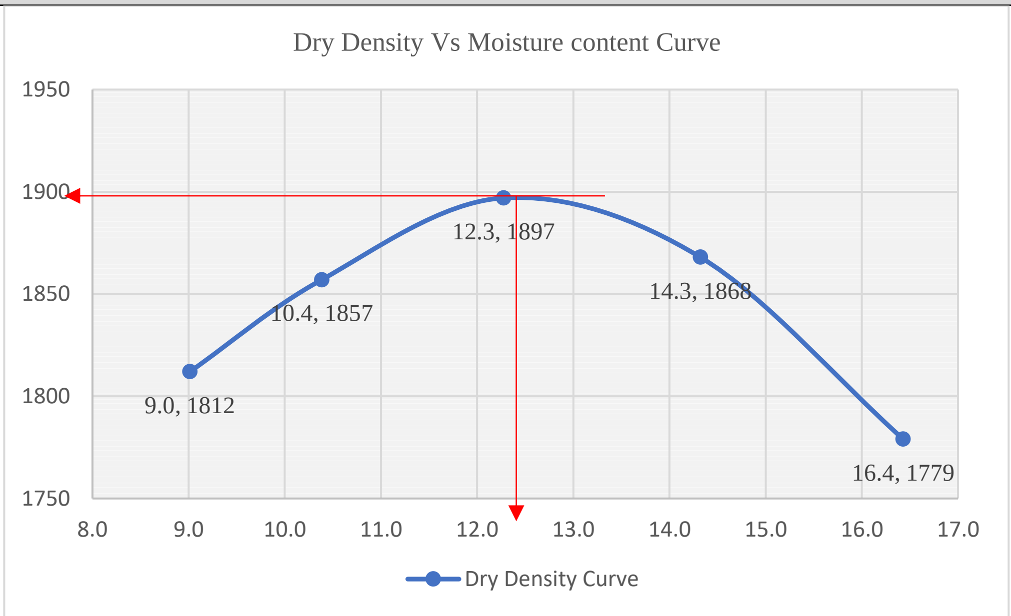
Bulk Density Determination

Test No.	**Unit**	**1**	**2**	**3**	**4**	**5**
Water added	g	130	230	330	430	530
Wt of mould+sample	g	9006.5	9214.5	9400.0	9410.5	9277.0
Wt of mould	g	4916	4916	4916	4916	4916
Wt of sample	g	4090.5	4298.5	4484	4494.5	4361
Volume of mould	cm^3	2105	2105	2105	2105	2105
Wet density	kg/m^3	1943	2042	2130	2135	2072
Dry density	**kg/m^3**	**1812**	**1857**	**1897**	**1868**	**1779**

Moisture Content Determination

Container No.	**Unit**	**TR**	**YFH**	**GDH**	**JSY**	**ZJT**	**RWT**	**UGS**	**IFS**	**SIY**	**ZJY**
Wt of wet soil + tin	g	334.2	334.5	332.5	332.1	429.0	408.5	391.0	402.5	404.0	400.3
Wt of dry soil + tin	g	307.2	314.4	306	305.4	388.0	368.6	347.5	357.4	353.0	349.6
Wt of moisture	g	27	20.1	26.5	26.7	41.0	39.9	43.5	45.1	51.0	50.7
Wt of tin	g	50.5	46.7	50.5	48.6	49.0	48.3	41.0	45.3	41.0	42.6
Wt of dry soil	g	256.7	267.7	255.5	256.8	339.0	320.3	306.5	312.1	312.0	307.0
Moisture content	%	10.5	7.5	10.4	10.4	12.1	12.5	14.2	14.5	16.3	16.5
Av. M/Content	**%**	**9.0**		**10.4**		**12.3**		**14.3**		**16.4**	

Compaction Curve



MDD:	**1899 kg/m^3**	**OMC:**	**12.4 %**

Table C.2-2: Field Density Test Summary for AP 108/15

GEOTECH SOLUTIONS	FIELD DENSITY TEST REPORT FOR AP 108/15		
Project:	Construction of structure pads along Karuma - Kawanda 400kV Transmission Line in Uganda.		
Client:	Samuel Acidri		
Test method:	BS 1377: Part 9: 1990	Testing Lab:	Geotech Solutions
Sample Ref.:	GEO/FDT/01-19/00001	Technician:	Herbert
Site Location:	AP 108/15	Checked by:	Bruce K

Depth of Test: (mm)	150		150	
Area of Test:	Foundation Pad		Foundation Pad	
Slide/Offset (m):	LHS		RHS	
Layer No.:	Top Layer - Fill		Top Layer - Fill	
Test No.:	1		2	
MOISTURE CONTENT DETERMINATION				
Container Number	**A1**	**A2**	**DS**	**RE**
Mass wet soil + container (g)	285.4	282.7	355.7	362.4
Mass dry soil + container (g)	254.2	250.6	324.6	332.2
Mass of container (g)	28.1	20.2	70.7	87.5
Mass of dry soil (g)	226.1	230.4	253.9	244.7
Mass of water (g)	31.2	32.1	31.1	30.2
Moisture content (%)	13.8	13.9	12.2	12.3
Average Moisture Content (%)	**13.9**		**12.3**	
DENSITY DETERMINATION				
Initial mass of sand (g)	8000.0		8000.0	
Final mass of sand (g)	2000.0		2000.0	
Mass of sand in cone (g)	1454.0		1454.0	
Mass of sand in hole (g)	4546.0		4546.0	
Density of sand (g/cm^3)	1.350		1.350	
Volume of the hole (cm^3)	3367.4		3367.4	
Mass of soil from the hole (g)	7970.6		7780.6	
In situ wet density (g/cm^3)	2.367		2.311	
In situ dry density (g/cm^3)	2.079		2.058	
Maximum dry density (g/cm^3)	2.062		2.062	
Optimum moisture content (%)	12.9		12.9	
Relative compaction (%)	**100.8**		**99.8**	

Remarks: These results relate to the sections that were tested.

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Table C.2-3: Field Density Test Summary for AP 108/20

GEOTECH SOLUTIONS	**FIELD DENSITY TEST REPORT FOR AP 108/20**			
Project:	Construction of structure pads along Karuma - Kawanda 400kV Transmission Line in Uganda.			
Client:	Samuel Acidri			
Test method:	BS 1377: Part 9: 1990	Testing Lab:	Geotech Solutions	
Sample Ref.:	GEO/SOILS/5-19/00051	Technician:	Herbert	
Site Location:	AP 108/20	Checked by:	Bruce K	

Depth of Test: (mm)	150		150	
Area of Test:	Structure Pad		Structure Pad	
Slide/Offset (m):	LHS		RHS	
Layer No.:	Top Layer - Fill		Top Layer - Fill	
Test No.:	1		2	
MOISTURE CONTENT DETERMINATION				
Container Number	**B1**	**A27**	**OP**	**XY**
Mass wet soil + container (g)	286.7	292.7	478.5	424.1
Mass dry soil + container (g)	264.0	268.7	441.7	393.8
Mass of container (g)	28.1	20.2	70.7	87.5
Mass of dry soil (g)	235.9	248.5	371.0	306.3
Mass of water (g)	22.7	24.0	36.8	30.3
Moisture content (%)	9.6	9.7	9.9	9.9
Average Moisture Content (%)	**9.6**		**9.9**	
DENSITY DETERMINATION				
Initial mass of sand (g)	7000.0		7000.0	
Final mass of sand (g)	1907.0		1740.0	
Mass of sand in cone (g)	1454.0		1454.0	
Mass of sand in hole (g)	3639.0		3806.0	
Density of sand (g/cm^3)	1.350		1.350	
Volume of the hole (cm^3)	2695.6		2819.3	
Mass of soil from the hole (g)	5191.0		5290.0	
In situ wet density (g/cm^3)	1.926		1.876	
In situ dry density (g/cm^3)	1.756		1.707	
Maximum dry density (g/cm^3)	1.846		1.846	
Optimum moisture content (%)	11.4		11.4	
Relative compaction (%)	**95.1**		**92.5**	

Remarks: These results relate to the sections that were tested.

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Laboratory Manager

Table C.2-4: Maximum Dry Density Test Summary for AP 108/20

Project:	Foundation along 400 kV Karuma - Kawanda Transmission Line.
Client:	Samuel Acidri/KPTL.

Maximum Dry Density (MDD) Test (BS 1377: Part 4: 1990)

Location:	AP 108/20	Volume, V (cm^3):	999.6
Mould dia. (mm):	105	Material Description:	Gravel material for pad foundations
Height of Mould (mm):	115.5	Technician/Engineer:	Shafik / Bruce K.
Number of Layers:	5	Blows per Layer:	27

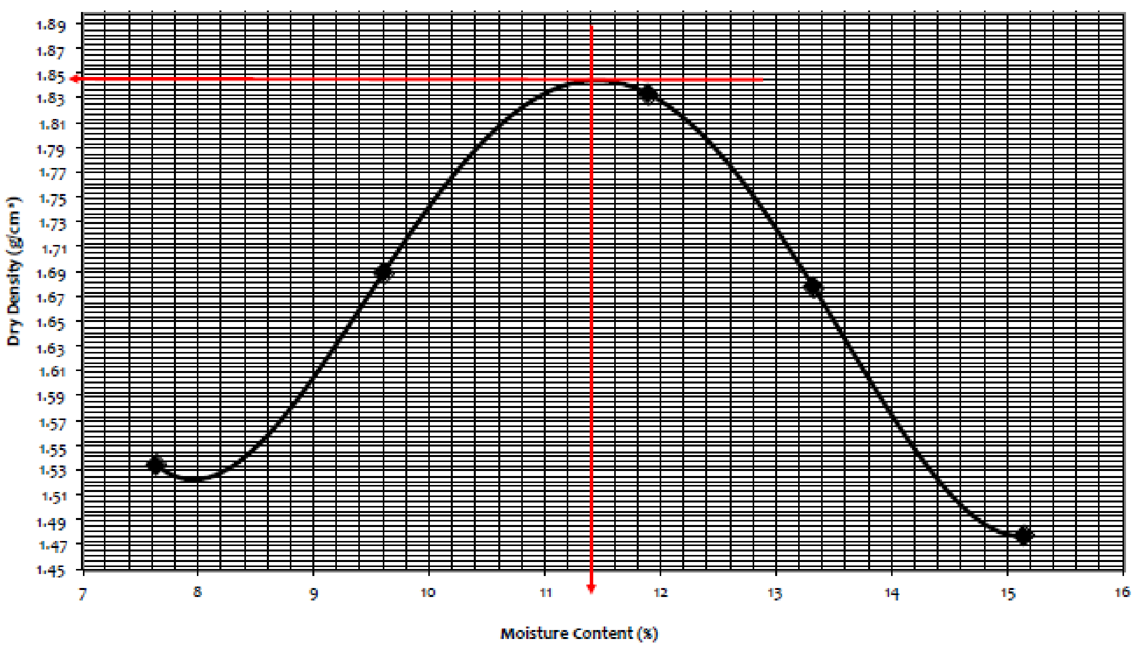
Bulk Density Determination

Test No.	**Unit**	**1**	**2**	**3**	**4**	**5**
Water used (V)	cm^3	100	200	300	400	500
Mould + base + sample (m_2)	g	5086	5286	5486	5336	5136
Mould + base (m_1)	g	3435	3435	3435	3435	3435
Compacted specimen (m_2-m_1)	g	1651	1851	2051	1901	1701
Bulk density, $\rho = (m_2-m_1)/V$	g/cm^3	1.65	1.85	2.05	1.90	1.70

Moisture Content Determination

Container No.	**Unit**	**BC**	**GH**	**OP**	**88**	**XX**	**IB**	**RC**	**LMN**	**CI**	**MI**
Container + wet soil	g	411.5	353.7	477.0	423.5	407.1	423.1	373.0	353.7	376.9	359.2
Dry soil + container	g	386.6	334.0	441.7	393.8	371.1	385.8	337.5	320.5	337.5	320.5
Container only	g	71.0	67.3	70.3	87.3	71.2	69.6	72.9	69.3	72.9	69.3
Moisture content	g	7.9	7.4	9.5	9.7	12.0	11.8	13.4	13.2	14.9	15.4
Ave. moisture content (W)	**g**	**7.6**		**9.6**		**11.9**		**13.3**		**15.1**	
Dry density, $P_d = (100\rho) / (100+W)$	**g/cm^3**	**1.53**		**1.69**		**1.834**		**1.68**		**1.48**	
	MDD	**1.846 g/cm^3**				**OMC**		**11.4 %**			

Compaction Curve



Appendix D - Static Load Test

Table D-1: AP 104/5 – Static Axial Uplift (Tensile) Load Test

Client: Samuel Acidri **Testing Firm:** Kalpataru Power Transmission Ltd **Date:** 11/07/2019

STATIC AXIAL UPLIFT (TENSILE) FOUNDATION LOAD TEST -FIELD RECORD SHEET

400 kV KARUMA-KAWANDA TRANSMISSION LINE PROJECT (UGANDA)

Type of Test: Static Axial Uplift (Tensile) Load Test **Foundation Location:** AP 104/5

Foundation Type: Waterlogged (Pile Foundation - 900 mm Diameter) **Site Weather:** Dry and Partly Cloudy

S/No.	Start Time	End Time	Elapsed Time	Load Step	Test Load		Waiting Period	Actual Load Indication	Foundation Displacement (Millimetres)						Remark
									Gauge 1 Reading (G1)			Gauge 2 Reading (G2)			
			(Minute)	(%)	kN	Ton	(Minute)	(Ton)	Initial	Middle	Final	Initial	Middle	Final	
1	11:02	11:12	10	10	55.57	5.7	10	5.7	0.00	0.00	0.00	0.01	0.04	0.04	18°C
2	11:14	11:24	10	25	138.92	14.2	10	15	0.00	0.00	0.00	0.06	0.05	0.02	19°C
3	11:26	11:36	10	50	277.85	28.3	10	30	0.00	0.00	0.00	0.08	0.07	0.07	19°C
4	11:37	11:47	10	70	388.98	39.7	10	40	0.04	0.07	0.10	0.12	0.13	0.16	19°C
5	11:48	11:58	10	80	444.55	45.3	10	45.3	0.14	0.16	0.19	0.18	0.21	0.23	20°C
6	11:59	12:09	10	90	500.12	51.0	10	51	0.22	0.20	0.15	0.26	0.24	0.20	22°C
7	12:10	12:40	30	**100**	**555.69**	**56.7**	30	57	0.19	0.11	**0.01**	0.22	0.15	**0.09**	24°C

Note:

After withstanding 100% of the design load for a waiting period of 30 minutes, the foundation is deemed to have successfully passed as per the International Standard EN 61773.

Table D-2: AP 104/5 – Lateral Load Test

Client: Samuel Acidri	**Testing Firm:** Kalpataru Power Transmission Ltd	**Date:**	12/07/2019

LATERAL FOUNDATION LOAD TEST -FIELD RECORD SHEET

400 kV KARUMA-KAWANDA TRANSMISSION LINE PROJECT (UGANDA)

Type of Test:	Lateral Load Test	**Foundation Location:**	AP 104/5
Foundation Type:	Waterlogged (Pile Foundation - 900 mm Diameter)	**Site Weather:**	Cloudy and Humid

S/No.	Start Time	End Time	Elapsed Time	Load Step	Test Load		Waiting Period	Actual Load Indication	Foundation Displacement (Millimetres)						Remark
									Gauge 1 Reading (G1)			Gauge 2 Reading (G2)			
			(Minute)	(%)	kN	Ton	(Minute)	(Ton)	Initial	Middle	Final	Initial	Middle	Final	
1	10:31	10:41	10	10	18.06	1.8	10	3	0.00	0.00	0.00	0.05	0.06	0.04	21°C
2	10:43	10:53	10	25	45.15	4.6	10	5	0.01	0.01	0.00	0.07	0.04	0.00	21°C
3	10:54	11:05	11	50	90.29	9.2	10	10	0.03	0.00	0.02	0.07	0.01	0.05	21°C
4	11:05	11:15	10	70	126.41	12.9	10	13	0.48	0.55	0.59	0.51	0.57	0.60	22°C
5	11:16	11:26	10	80	144.47	14.7	10	15	0.88	0.89	0.88	0.88	0.88	0.86	22°C
6	11:26	11:37	11	90	162.53	16.6	10	17	1.34	1.46	1.49	1.31	1.43	1.45	22°C
7	11:37	12:08	31	**100**	**180.59**	**18.4**	30	19	2.18	2.39	**2.39**	2.13	2.34	**2.35**	22°C

Note:

After withstanding 100% of the design load for a waiting period of 30 minutes, the foundation is deemed to have successfully passed as per the International Standard EN 61773.

Table D-3: AP 104/5 – Compressive Load Test

Client:	Samuel Acidri	**Testing Firm:**	Kalpataru Power Transmission Ltd	**Date:**	15/07/2019

COMPRESSIVE FOUNDATION LOAD TEST -FIELD RECORD SHEET

400 kV KARUMA-KAWANDA TRANSMISSION LINE PROJECT (UGANDA)

Type of Test:	Compressive Load Test	**Foundation Location:**	AP 104/5
Foundation Type:	Waterlogged (Pile Foundation - 900 mm Diameter)	**Site Weather:**	Cloudy and Rainy

S/No.	**Start Time**	**End Time**	**Elapsed Time**	**Load Step**	**Test Load**		**Waiting Period**	**Actual Load Indication**	**Foundation Displacement (Millimetres)**						**Remark**
									Gauge 1 Reading (G1)			**Gauge 2 Reading (G2)**			
			(Minute)	**(%)**	**kN**	**Ton**	**(Minute)**	**(Ton)**	**Initial**	**Middle**	**Final**	**Initial**	**Middle**	**Final**	
1	13:39	13:49	10	10	82.85	8.4	10	9	0.00	-0.24	-0.24	0.00	-0.21	-0.24	25°C
2	13:52	14:02	10	25	207.13	21.1	10	22	-0.30	-0.31	-0.53	-0.30	-0.33	-0.42	28°C
3	14:06	14:16	10	50	414.26	42.2	10	43	-1.04	-1.16	-1.23	-0.65	-0.74	-0.83	29°C
4	14:18	14:28	10	70	579.96	59.1	10	60	-1.24	-1.37	-1.36	-0.83	-0.93	-0.94	29°C
5	14:29	14:39	10	80	662.82	67.6	10	68	-1.26	-0.91	-0.80	-0.90	-0.68	-0.62	28°C
6	14:40	14:50	10	90	745.67	76.0	10	76	-0.79	-0.76	-0.67	-0.63	-0.64	-0.56	21°C
7	14:51	15:21	30	100	828.52	84.5	30	85	-0.69	-0.15	-0.62	-0.55	-0.24	-0.41	19°C
8	15:23	15:26	3	110	911.37	92.9	3	93	-0.67	-0.67	-0.68	-0.45	-0.48	-0.49	19°C
9	15:27	15:30	3	120	994.22	101	3	101	-0.71	-0.73	-0.73	-0.52	-0.54	-0.54	19°C
10	15:31	15:34	3	**130**	**1077.08**	**110**	3	110	-0.80	-0.82	**-0.83**	-0.58	-0.59	**-0.60**	19°C

Note:
After withstanding 100% of the design load for 30 minutes waiting period, the foundation is deemed to have successfully passed as per EN 61773.

Table D-4: AP 108/20 – Static Axial Uplift (Tensile) Load Test

Client: Samuel Acidri **Testing Firm:** Kalpataru Power Transmission Ltd **Date:** 05/05/2019

STATIC AXIAL UPLIFT (TENSILE) FOUNDATION LOAD TEST -FIELD RECORD SHEET

400 kV KARUMA-KAWANDA TRANSMISSION LINE PROJECT (UGANDA)

Type of Test: Static Axial Uplift (Tensile) Load Test **Foundation Location:** AP 108/20

Foundation Type: Good Soil (Murram soil location) **Site Weather:** Dry and Sunny

S/No.	Start Time	End Time	Elapsed Time	Load Step	Test Load		Waiting Period	Actual Load Indication	Foundation Displacement (Millimetres)						Remark
									Gauge 1 Reading (G1)			Gauge 2 Reading (G2)			
			(Minute)	(%)	kN	Ton	(Minute)	(Ton)	Initial	Middle	Final	Initial	Middle	Final	
1	2:17	2:27	10	10	59.45	6.1	10	6.1	0.00	0.00	0.00	0.00	0.00	0.01	26°C
2	2:28	2:38	10	25	148.61	15.2	10	15.2	-0.03	-0.04	-0.05	0.00	0.01	0.01	27°C
3	2:40	2:50	10	50	297.23	30.3	10	30.3	-0.17	-0.17	-0.17	-0.04	-0.04	-0.03	29°C
4	2:51	3:01	10	70	416.12	42.4	10	42.5	-0.27	-0.30	-0.30	-0.02	-0.02	-0.02	30°C
5	3:02	3:12	10	80	475.56	48.5	10	48.5	-0.37	-0.38	-0.40	0.00	0.03	0.05	31°C
6	3:13	3:23	10	90	535.01	54.6	10	54.6 (1)	-0.56	-0.58	-0.61	0.13	0.15	0.14	31°C
7	3:25	3:55	30	**100**	**594.45**	**60.6**	30	61.0 (2)	-0.77	-0.86	**-0.87**	0.11	0.18	**0.19**	30°C
8	3:57	4:00	3	110	653.90	66.7	3	67	-1.07		-1.08	0.14		0.13	29°C
9	4:01	4:04	3	120	713.34	72.7	3	73	-1.26		-1.30	0.05		0.04	29°C
10	4:05	4:08	1	130	772.79	78.8	3	80	-1.64			-0.21			28°C

Note:

(1) At 90% design load, pressure on the jack reduced and the load values reached approx. 53MT instead of 54.5MT.

(2) At 100% design load, pressure on the jack reduced and the load values reached approx. 60MT instead of 60.6MT after 15 minutes waiting period. Hence, the load on site was increased to 63MT and kept it balancing for the remaining 15 minutes waiting period.

Table D-5: AP 108/15 – Static Axial Uplift (Tensile) Load Test

Client: Samuel Acidri **Testing Firm:** Kalpataru Power Transmission Ltd **Date:** 22/05/2019

STATIC AXIAL UPLIFT (TENSILE) FOUNDATION LOAD TEST -FIELD RECORD SHEET

400 kV KARUMA-KAWANDA TRANSMISSION LINE PROJECT (UGANDA)

Type of Test: Static Axial Uplift (Tensile) Load Test **Foundation Location:** AP 108/15

Foundation Type: ST-Poor Soil -Dry (Black cotton soil location) **Site Weather:** Dry and Sunny

S/No.	Start Time	End Time	Elapsed Time (Minute)	Load Step (%)	Test Load kN	Test Load Ton	Waiting Period (Minute)	Actual Load Indication (Ton)	Foundation Displacement (Millimetres) Gauge 1 Reading (G1) Initial	G1 Middle	G1 Final	Gauge 2 Reading (G2) Initial	G2 Middle	G2 Final	Remark
1	10:47	10:57	10	10	72.72	7.4	10	8	-0.0010	-	-0.0010	-0.02	-	-0.02	25°C
2	10:59	11:09	10	25	181.80	18.5	10	19	-0.0010	-0.0005	-0.0005	-0.02	-0.02	-0.02	25°C
3	11:13	11:23	10	50	363.60	37.1	10	38	-0.0005	0.0020	0.0020	-0.02	0.02	0.01	25°C
4	11:26	11:36	10	70	509.04	51.9	10	52	0.002	0.0065	0.0075	0.01	0.13	0.15	26°C
5	11:38	11:48	10	80	581.76	59.3	10	60	0.0075	0.0110	0.0120	0.15	0.25	0.30	26°C
6	11:49	11:59	10	90	654.48	66.7	10	67	0.0120	0.0135	0.0130	0.30	0.35	0.33	27°C
7	12:00	12:30	30	100	727.20	74.2	30	75	0.0130	0.0160	0.0190	0.33	0.41	0.44	27°C
8	12:32	12:35	3	110	799.92	81.6	3	82	0.0190	0.0210	0.0220	0.44	0.52	0.54	28°C
9	12:36	12:39	3	120	872.64	89.0	3	90	0.0220	0.0270	0.0285	0.54	0.68	0.70	28°C
10	12:40	12:43	3	**130**	**945.36**	**96.4**	3	97	0.0285	0.0325	**0.0335**	0.70	0.80	**0.83**	28°C
12	12:45	12:48	3	140	1018.08	103.9	3	104	0.0335	0.0410	0.0415	0.83	1.00	1.03	29°C
13	12:49	12:52	3	150	1090.80	111.3	3	112	0.0415	0.0455	0.0455	1.03	1.14	1.14	29°C

Note:

(1) Minor cracks were observed along the micrometre G1 face at 70% (20cm crack along the stub), at 80% (3cm long crack at top of chimney), at 90% (9cm long crack top of chimney) and at 100% (15cm long crack). Along the Micrometre G2 face, cracks were observed at 80% (0cm along stub), at 90% (8cm long), at 100% (16cm long), and at 110% (16cm long on top of chimney and 20cm on the side).

(2) Severe cracks were observed between 120% and 150%, with minor cracks on the backfilled and compacted soil; and minor pressure drops.

Table D-6: KL 30 (B103+5) – Calibration and Loading sequence details

Load gauge calibration and conversions	
Foundation Testing Date:	22/04/2019
Load:	Oil pressure x active area of piston
Oil pressure 100 bar:	1 kN/cm^2
Active piston Area, A:	729.9 cm^2
Therefore 1 kN load yields:	1/729.9 in kN/cm^2 (100 bars) using one jack
Design Load = 962.26 kN	(962.26 x 0.137) bars = 131.79 bars
Target Load = 1250.94 kN	(1250.94 x 0.137) bars = 171.36 bars

Load steps percentage (%)	**Load steps (kN)**	**Load steps (Tonnes)**	**Pressure pump reading (bars)**
25	240.5	24.1	32.95
50	481.0	48.1	65.89
70	673.4	67.3	92.26
80	769.6	77.0	105.44
90	865.8	86.6	118.61
100	962.26	96.2	131.79
110	1058.2	105.8	144.97
120	1154.4	115.4	158.15
130	1250.9	125.1	171.36

Loading sequence				
Load			**Reading time interval**	**Cumulative time**
% of design load	**kN**	**Ton**	**min**	**min**
25	240.5	13.03	0	5
			5	10
			10	15
50	481.0	48.1	0	20
			5	25
			10	30
70	673.4	67.3	0	35
			5	40
			10	45

80	769.6	77	0	50
			5	55
			10	60
90	865.8	86.6	0	65
			5	70
			10	75
100	962.3	96.2	0	80
			5	85
			10	90
			20	100
			30	110
110	1058.2	105.8	0	115
			5	118
120	1154.4	115.4	0	123
			5	126
130	1250.9	125.1	0	131
			5	134

Table D-7: KL 30 (B103+5) – Record of Load Testing Readings

Axial Uplift Load Test Report for KL 30 (IEC 1773: 1996)					
Project:	Axial Uplift Load Test Foundation (132 kV Karuma-Lira Transmission Line)				
Foundation Type:	DB-Waterlogged Location	**Foundation Base Size:**	4.5m x 4.5m	**Testing Date:**	22/04/2019
Coordinate:	N 426605.777, E 246178.091	**Client:**	Samuel Acidri / Sinohydro Corporation Ltd		
Design Load:	962.26 kN (96.2 Ton)	**Weather:**	Sunny / cloudy and occasional winds		
Target Load:	1250.94 kN (125.1 Ton)	**Testing Firm:**	COMATLAB (U) Ltd		

No.	% of design load	Test Load (kN)	Indication Load (oil pressure, bars)	Elapsed time	Displacement Gauge A (mm)	Gauge B (mm)	Gauge D (mm)	Gauge A disp. (mm)	Gauge B disp. (mm)	Gauge D disp. (mm)	Temp (°C)	Load Cell	Remarks
1	10%	96.2	13.18	0	18.00	28.86	38.88				28.6	0.00135	
2	25%	240.5	32.95	5	18.00	29.25	39.34	0	-0.39	-0.46	28.7	0.00335	
				10	18.00	28.87	38.95	0	0.38	0.39	28.6	0.00334	
				15			39.55	-1	-0.03	-0.6	28.6		Ref. beam interference
3	50%	481.0	65.89	20	19.52	29.09	39.55	-0.52	-0.19	0	29.3	0.00670	
				25	19.57	28.59	39.42	0.05	0.5	0.13	28.7	0.00612	
				30	19.45	28.11	38.12	0.12	0.48	1.3	29.9	0.01390	
4	70%	673.4	92.26	35	18.86	26.43	37.25	0.59	1.68	0.87	29.7	0.01390	
				40	18.62	27.70	37.75	0.24	-1.27	-0.5	30.0	0.01385	Strong wind
				45	17.12	26.68	37.60	1.5	1.02	0.15	28.9	0.01380	
5	80%	769.6	105.44	50	16.02	25.50	36.48	1.1	1.18	1.12	28.9	0.0140	
				55	16.60	26.45	36.42	-0.58	-0.95	0.06	29.9	0.0140	
				60	15.67	25.11	35.55	0.93	1.34	0.87	29.4	0.0140	
6	90%	865.8	118.61	65	14.85	23.53	34.30	0.82	1.58	1.25	28.6	0.01537	
				70	14.71	23.44	34.18	0.14	0.09	0.12	28.1	0.01537	
				75	13.68	23.43	34.18	1.03	0.01	0	28.2	0.01536	
7	100%	962.0	131.79	80	14.41	23.19	33.92	-0.73	0.24	0.26	28.2	0.01637	
				85	14.40	23.19	33.92	0.01	0	0	28.6	0.01535	
				90	14.41	23.23	33.93	-0.01	-0.04	-0.01	28.5	0.01635	
				95	14.75	23.65	34.55	-0.34	-0.42	-0.62	28.1	0.01635	
				100	16.52	26.79	37.90	-1.77	-3.14	-3.35	28.3	0.01635	Strong Wind
				105	15.76	25.65	35.40	-1.35	-2.42	2.5	28.1	0.01634	
				110	15.48	25.48	36.31	0.28	0.17	-0.91	28.1	0.01634	
8	110%	1058.2	144.97	115	15.35	25.3	34.9	0.13	0.18	1.41	28.0	0.0160	Slight Wind
				118	14.62	25.59	34.4	0.73	-0.29	0.5	26.6	0.0160	
9	120%	1154.4	158.15	123	14.53	25.59	34.25	0.09	0	0.15	27.1	0.0175	
				126	14.52	24.71	34.25	0.01	0.88	0	26.9	0.0175	
10	130%	1250.6	171.36	131	16.3	27.65	36.6	-1.78	-2.94	-2.35	28.4	0.0160	
				134	16.71	26.99	36.2	-0.41	0.66	0.4	28.7	0.0179	

Table D-8: KL 30 (B103+5) – Determination of Displacement Readings

% of design load	Load (kN)	Time (min)	Gauge A disp. (mm)	Gauge B disp. (mm)	Gauge D disp. (mm)	Ave. disp. (mm)	Cumulative disp. (mm)	Maximum disp. (mm)
25	240.5	5	0	-0.39	-0.46	-0.28	-0.28	-0.03
		10	0	0.38	0.39	0.26	-0.03	
		15	-1	-0.03	-0.6	-0.54	-0.57	
50	481.0	20	-0.52	-0.19	0	-0.24	-0.81	0.02
		25	-0.05	0.5	0.13	0.19	-0.61	
		30	0.12	0.48	1.3	0.63	0.02	
70	673.4	35	0.59	1.68	0.87	1.05	1.07	1.45
		40	0.24	-1.27	-0.5	-0.51	0.56	
		45	1.5	1.02	0.15	0.89	1.45	
80	769.6	50	1.1	1.18	1.12	1.13	2.58	3.14
		55	-0.58	-0.95	0.06	-0.49	2.09	
		60	0.93	1.34	0.87	1.05	3.14	
90	865.8	65	0.82	1.58	1.25	1.22	4.35	4.47
		70	0.14	0.09	0.12	0.12	4.47	
		75	1.03	0.01	0	0.35	4.82	
100	962.3	80	-0.73	0.24	0.26	-0.08	4.74	4.74
		85	0.01	0	0	0.00	4.74	
		90	-0.01	-0.04	-0.01	-0.02	4.72	
		95	-0.34	-0.42	-0.62	-0.46	4.26	
		100	-1.77	-3.14	-3.35	-2.75	1.51	
		105	-1.35	-2.42	2.5	-0.42	1.09	
		110	0.28	0.17	-0.91	-0.15	0.93	
110	1058.2	115	0.13	0.18	1.41	0.57	4.84	5.15
		118	0.73	-0.29	0.5	0.31	5.15	
120	1154.4	123	0.09	0	0.15	0.08	5.23	5.53
		126	0.01	0.88	0	0.30	5.53	
130	1250.6	131	-1.78	-2.94	-2.35	-2.36	5.74	5.96
		134	-0.41	0.66	0.4	0.22	5.96	

Table D-9: KL 30 (B103+5) - Analysis Table

Axial Uplift Load Test Report for KL 30 (IEC 1773: 1996)			
Project:	Axial Uplift Load Test Foundation (132 kV Karuma-Lira Transmission Line)		
Foundation Type:	DB-Waterlogged Location	**Foundation Base Size:**	4.5m x 4.5m
Coordinate:	N 426605.777, E 246178.091	**Depth:**	-4.5m
Design Load:	962.26 kN (96.2 Ton)	**Weather:**	Sunny/ cloudy and occasional winds
Target Load:	1250.94 kN (125.1 Ton)	**Gauge:**	A, B & D

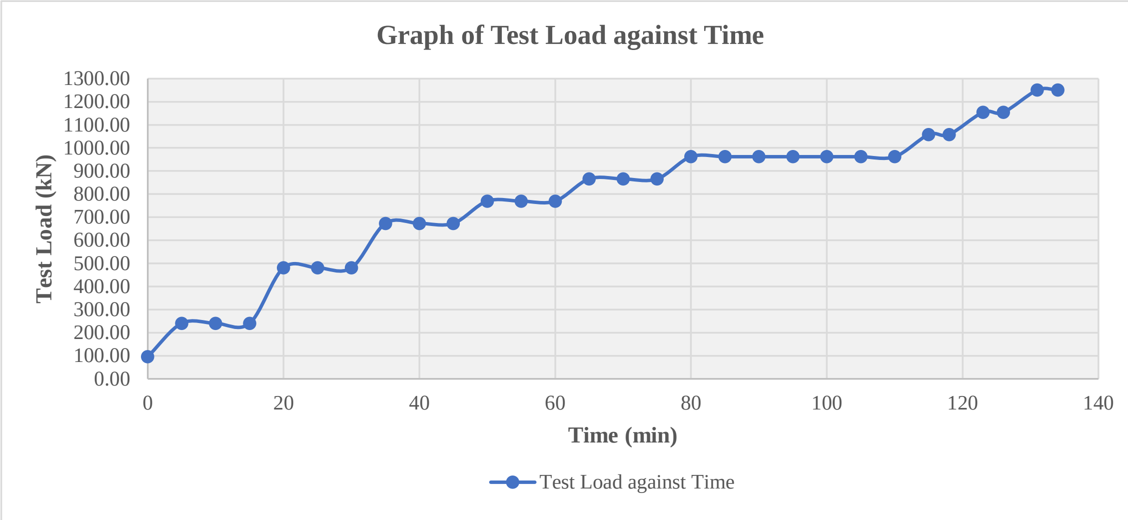
No.	**% of design load**	**Test Load (kN)**	**Indication Load (bars)**	**Elapsed time**	**Displacement**									**Load Cell**	**Remarks**
					Gauge A (mm)	**Gauge B (mm)**	**Gauge D (mm)**	**Gauge A disp. (mm)**	**Gauge B disp. (mm)**	**Gauge D disp. (mm)**	**Ave. disp. (mm)**	**Cumm. Disp. (mm)**	**Max. disp. (mm)**		
1	10%	96.2	13.18	0	18.00	28.86	38.88	0	0	0	0	0	0	0.00135	
2	25%	240.5	32.95	5	18.00	29.25	39.34	0	-0.39	-0.46	-0.28	-0.28	-0.03	0.00335	
				10	18.00	28.87	38.95	0	0.38	0.39	0.26	-0.03		0.00334	
				15	19.00		39.55	-1	-0.03	-0.6	-0.54	-057			Ref. beam interference
3	50%	481.0	65.89	20	19.52	29.09	39.55	-0.52	-0.19	0	-0.24	-0.81	0.02	0.00670	
				25	19.57	28.59	39.42	0.05	0.5	0.13	0.19	-0.61		0.00612	
				30	19.45	28.11	38.12	0.12	0.48	1.3	0.63	0.02		0.01390	
4	70%	673.4	92.26	35	18.86	26.43	37.25	0.59	1.68	0.87	1.05	1.07	1.45	0.01390	
				40	18.62	27.70	37.75	0.24	-1.27	-0.5	-0.51	0.56		0.01385	Strong wind
				45	17.12	26.68	37.60	1.5	1.02	0.15	0.89	1.45		0.01380	

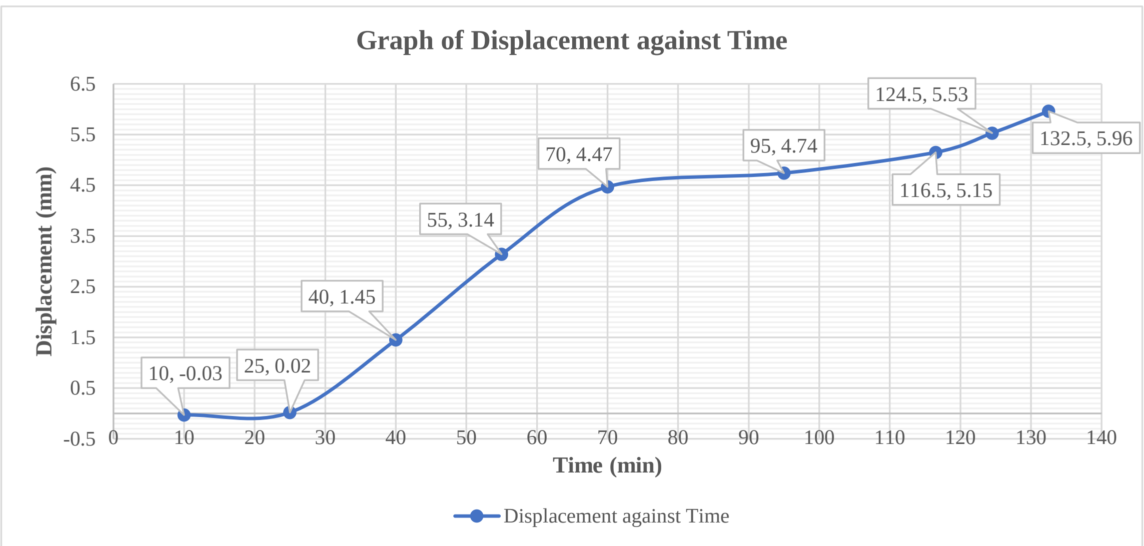
5	80%	769.6	105.44	50	16.02	25.50	36.48	1.1	1.18	1.12	1.13	2.58	3.14	0.0140	
				55	16.60	26.45	36.42	-0.58	-0.95	0.06	-0.49	2.09		0.0140	
				60	15.67	25.11	35.55	0.93	1.34	0.87	1.05	3.14		0.0140	
6	90%	865.8	118.61	65	14.85	23.53	34.30	0.82	1.58	1.25	1.22	4.35	4.47	0.01537	
				70	14.71	23.44	34.18	0.14	0.09	0.12	0.12	4.47		0.01537	
				75	13.68	23.43	34.18	1.03	0.01	0	0.35	4.82		0.01536	
7	100%	962.0	131.79	80	14.41	23.19	33.92	-0.73	0.24	0.26	-0.08	4.74	4.74	0.01637	
				85	14.40	23.19	33.92	0.01	0	0	0.00	4.74		0.01535	
				90	14.41	23.23	33.93	-0.01	-0.04	-0.01	-0.02	4.72		0.01635	
				95	14.75	23.65	34.55	-0.34	-0.42	-0.62	-0.46	4.26		0.01635	
				100	16.52	26.79	37.90	-1.77	-3.14	-3.35	-2.75	1.51		0.01635	Strong Wind
				105	15.76	25.65	35.40	-1.35	-2.42	2.5	-0.42	1.09		0.01634	
				110	15.48	25.48	36.31	0.28	0.17	-0.91	-0.15	0.93		0.01634	
8	110%	1058.2	144.97	115	15.35	25.3	34.9	0.13	0.18	1.41	0.57	4.84	5.15	0.0160	Slight Wind
				118	14.62	25.59	34.4	0.73	-0.29	0.5	0.31	5.15		0.0160	
9	120%	1154.4	158.15	123	14.53	25.59	34.25	0.09	0	0.15	0.08	5.23	5.53	0.0175	
				126	14.52	24.71	34.25	0.01	0.88	0	0.30	5.53		0.0175	
10	130%	1250.6	171.36	131	16.3	27.65	36.6	-1.78	-2.94	-2.35	-2.36	5.74	5.96	0.0160	
				134	16.71	26.99	36.2	-0.41	0.66	0.4	0.22	5.96		0.0179	

Table D-10: KL 30 (B103+5) – Hyperbolic analysis method

% of design load	Load (kN)	Time (min)	Maximum displacement (mm)	Displacement/Load [(mm/kN) x 10^{-3}]
25	240.5	5 10 15	-0.03 = 0.0	0.00
50	481.0	20 25 30	0.02	0.04158
70	673.4	35 40 45	1.45	2.153252
80	769.6	50 55 60	3.14	4.080042
90	865.8	65 70 75	4.47	5.162855
100	962.3	80 85 90 95 100 105 110	4.74	4.927235
110	1058.2	115 118	5.15	4.866755
120	1154.4	123 126	5.53	4.790367
130	1250.6	131 134	5.96	4.765712

KL 30 STATIC LOAD TEST GRAPHS





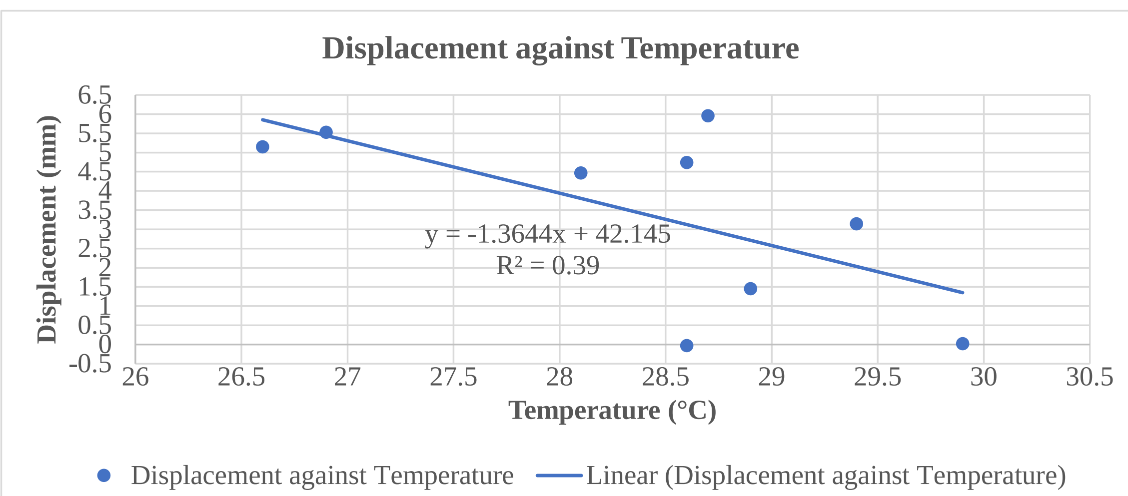
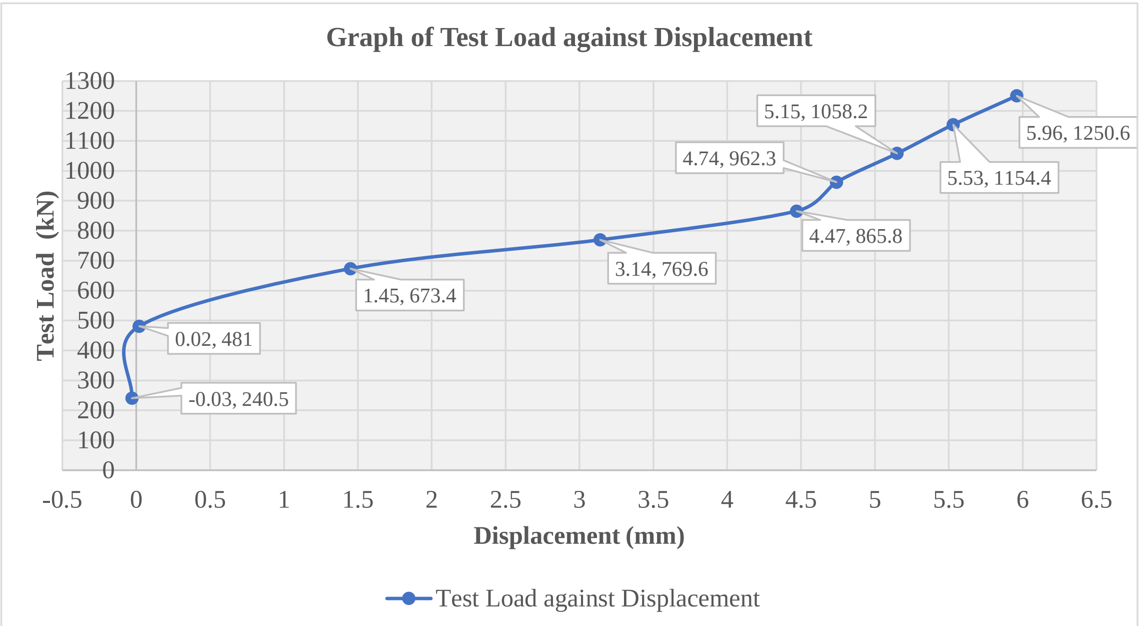


Fig. D-1 (a): KL 30- Static Load test Graphs

KL 30 STATIC LOAD TEST GRAPHS



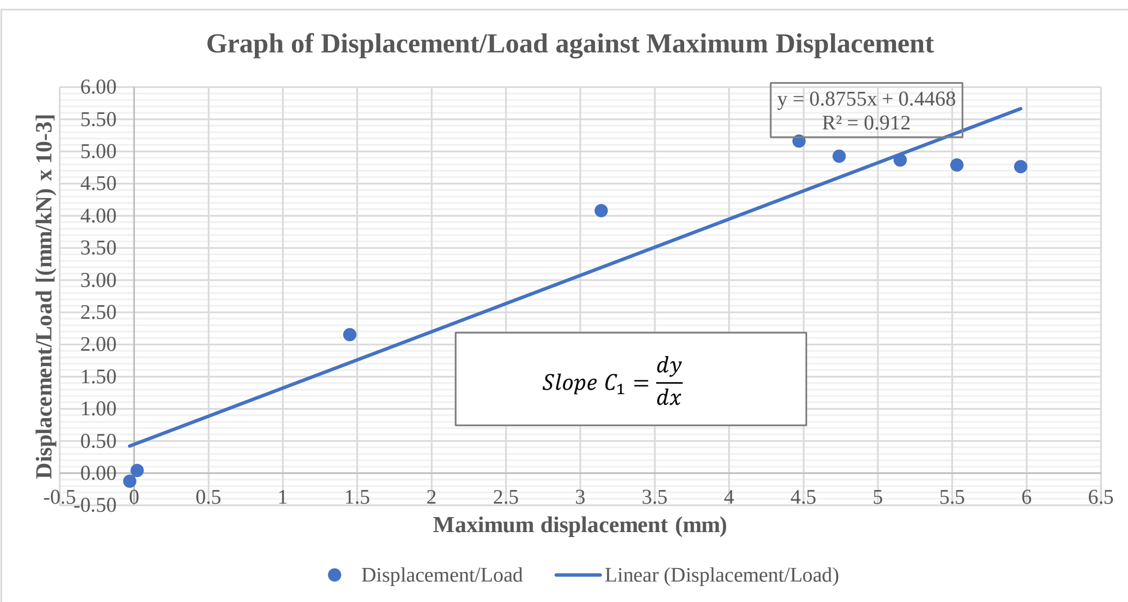
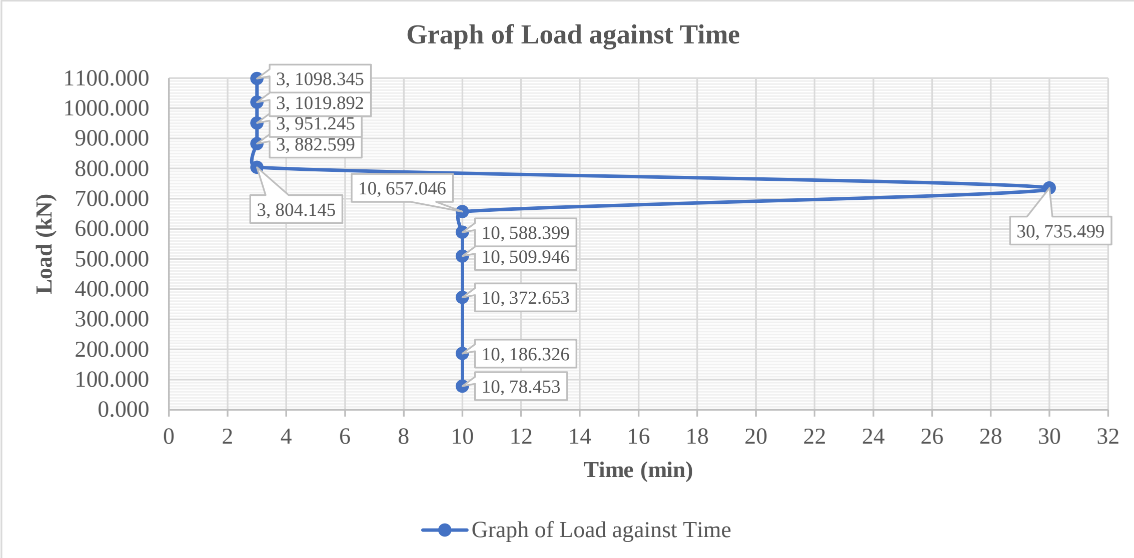
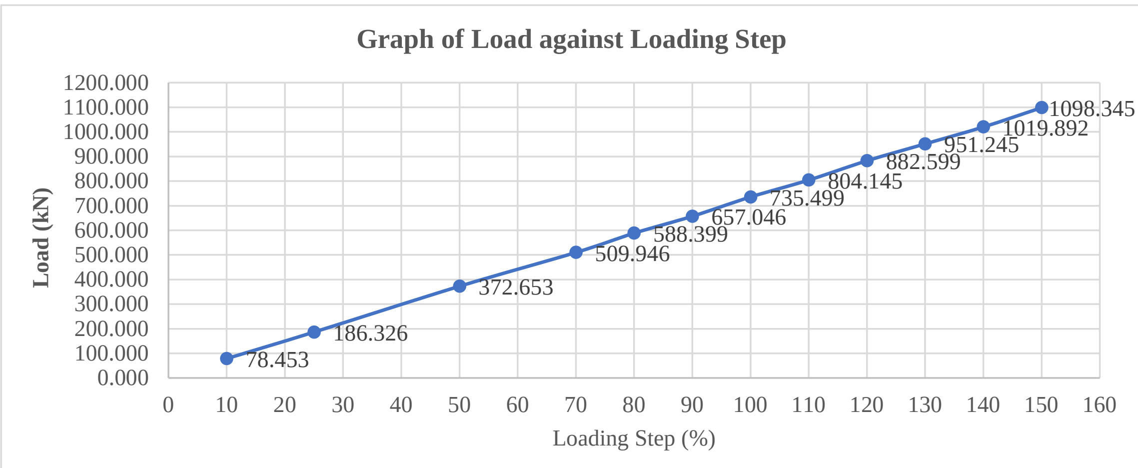


Fig. D-1 (b): KL 30- Static Load test Graphs

AP 108/15 STATIC LOAD TEST GRAPHS





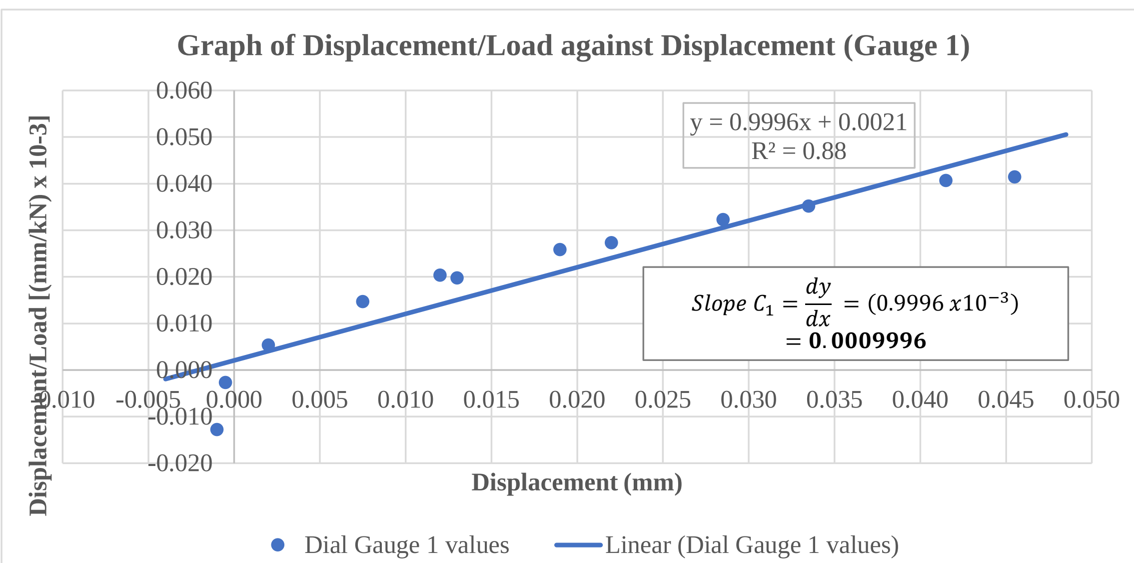
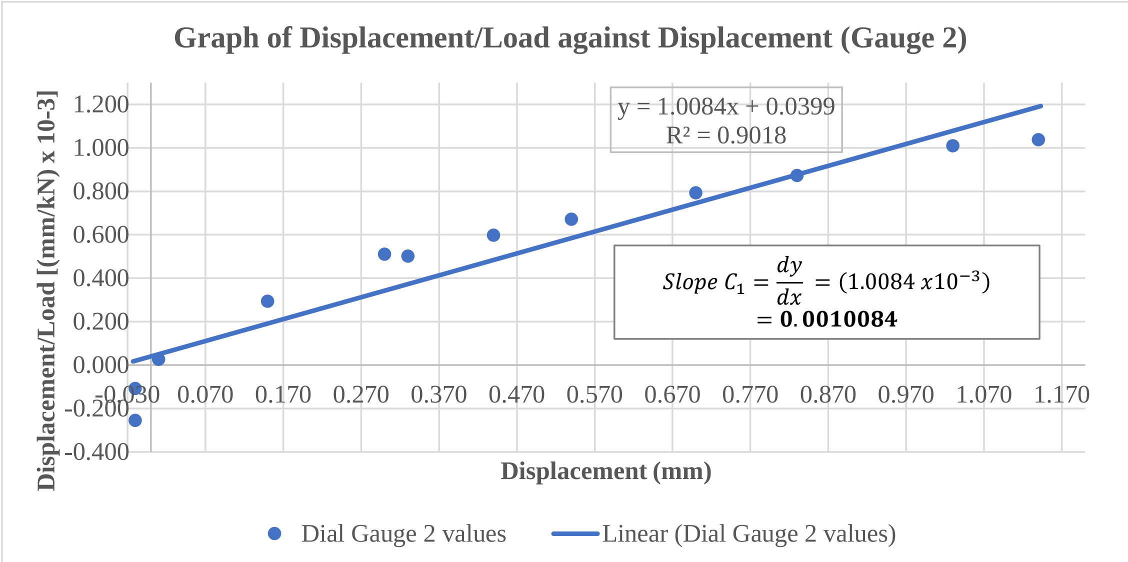
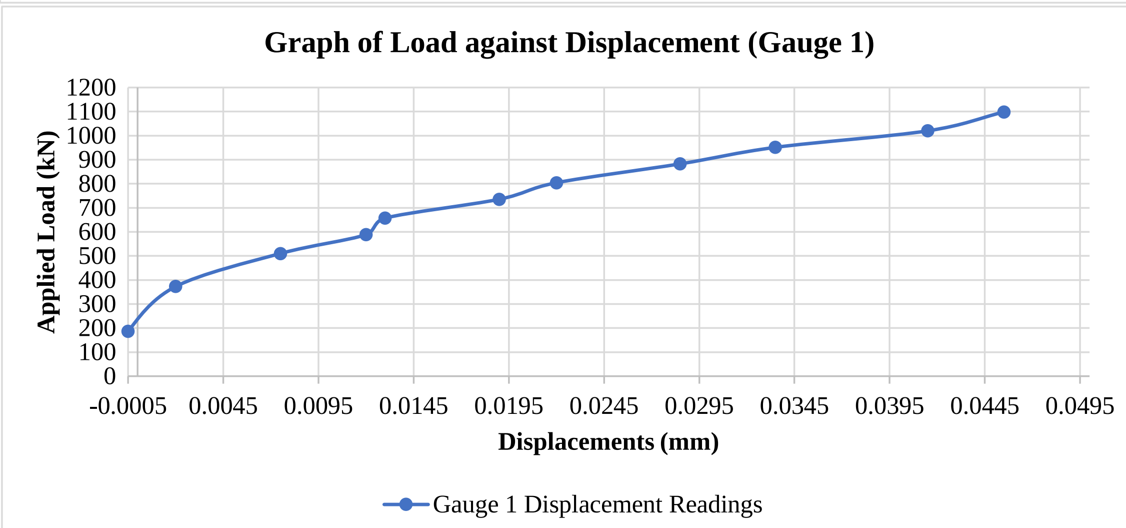


Fig. D-2 (a): AP 108/15- Static Load test Graphs

AP 108/15 STATIC LOAD TEST GRAPHS





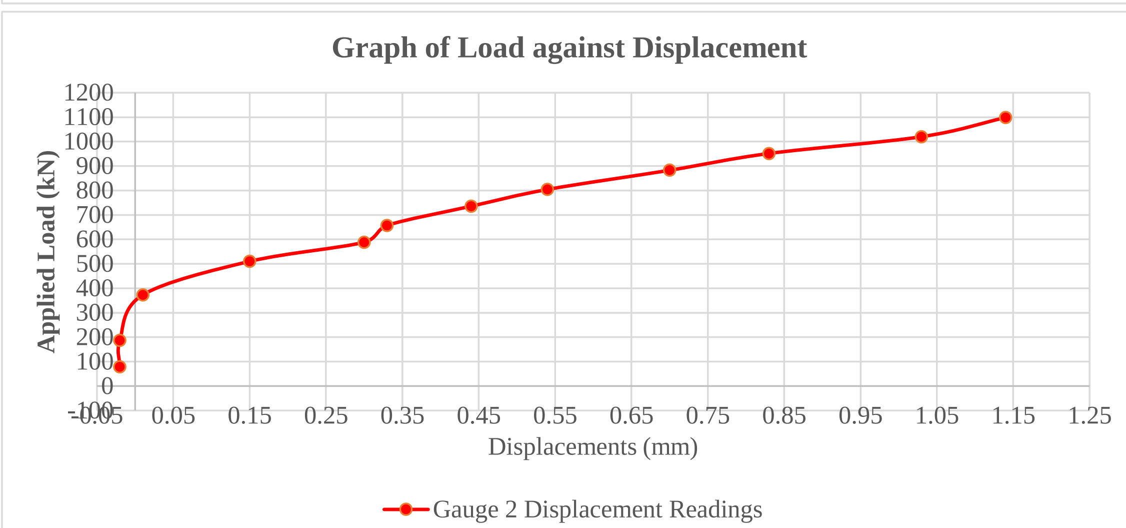
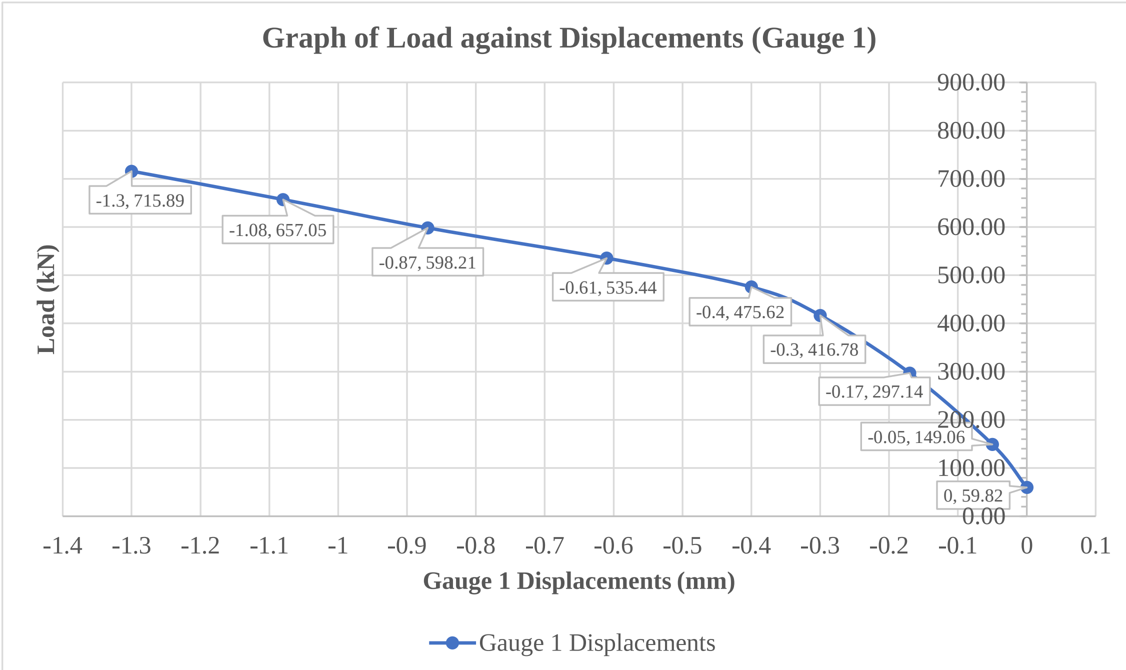


Fig. D-2 (b): AP 108/15- Static Load test Graphs

AP 108/20 STATIC LOAD TEST GRAPHS



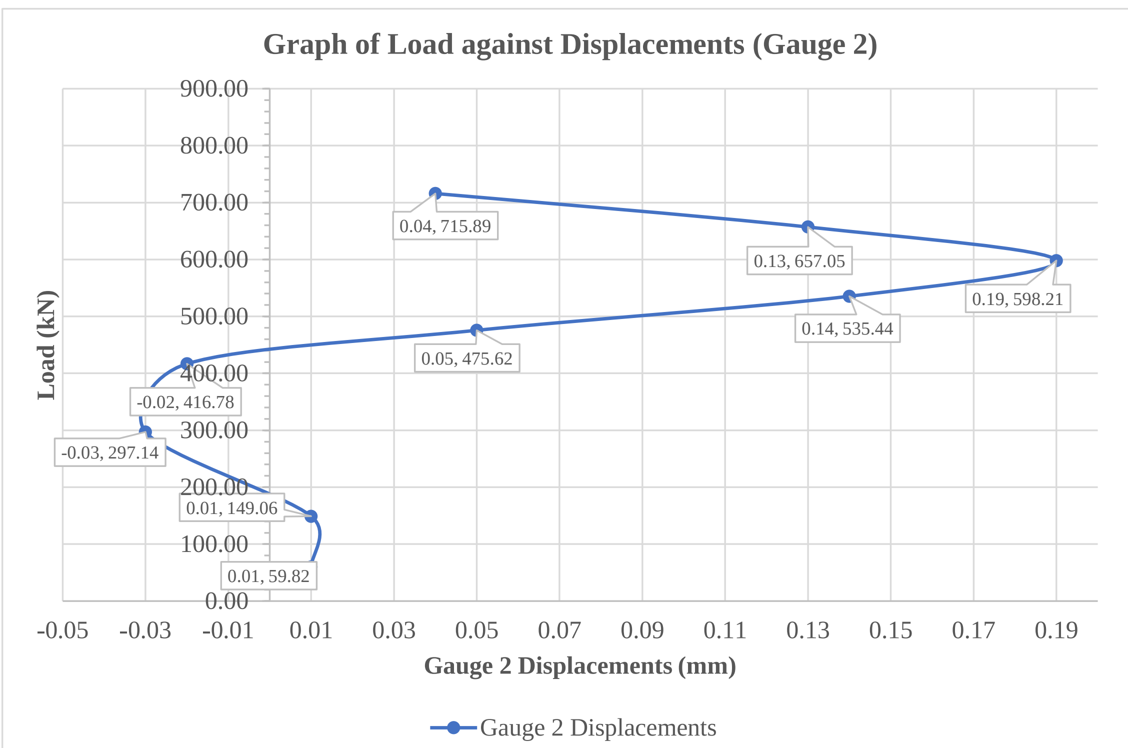
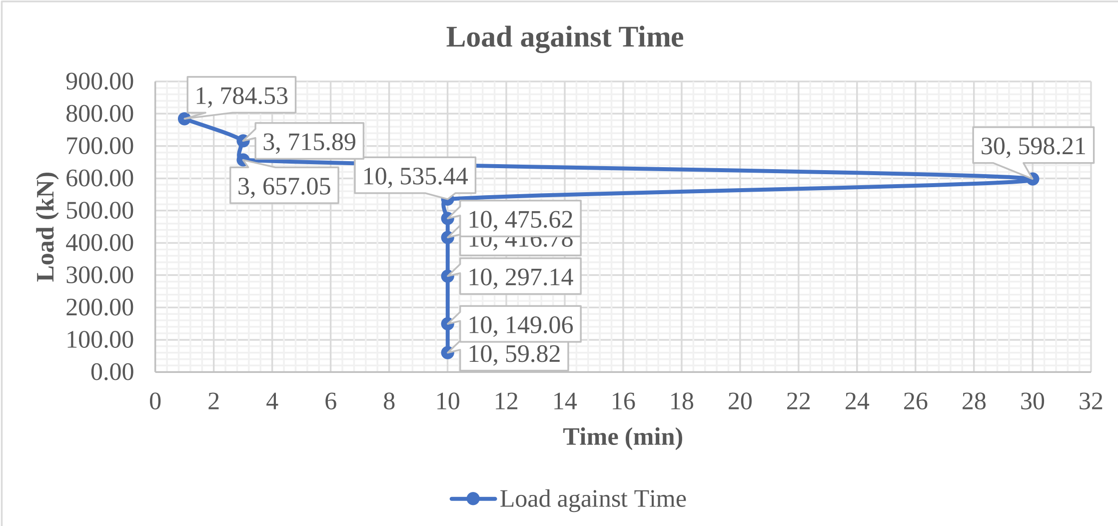
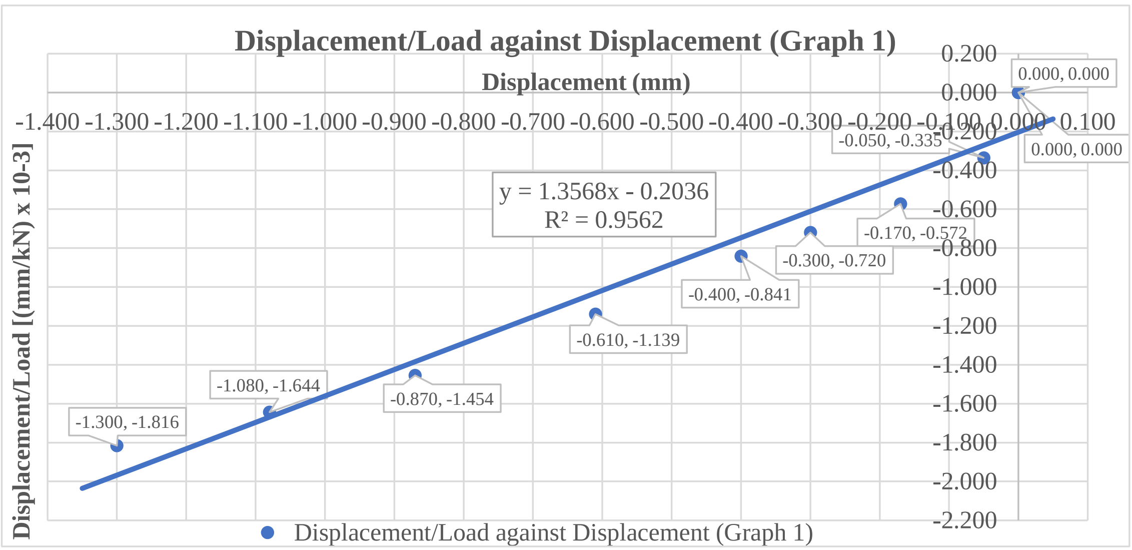


Fig. D-3 (a): AP 108/20- Static Load test Graphs

AP 108/20 STATIC LOAD TEST GRAPHS





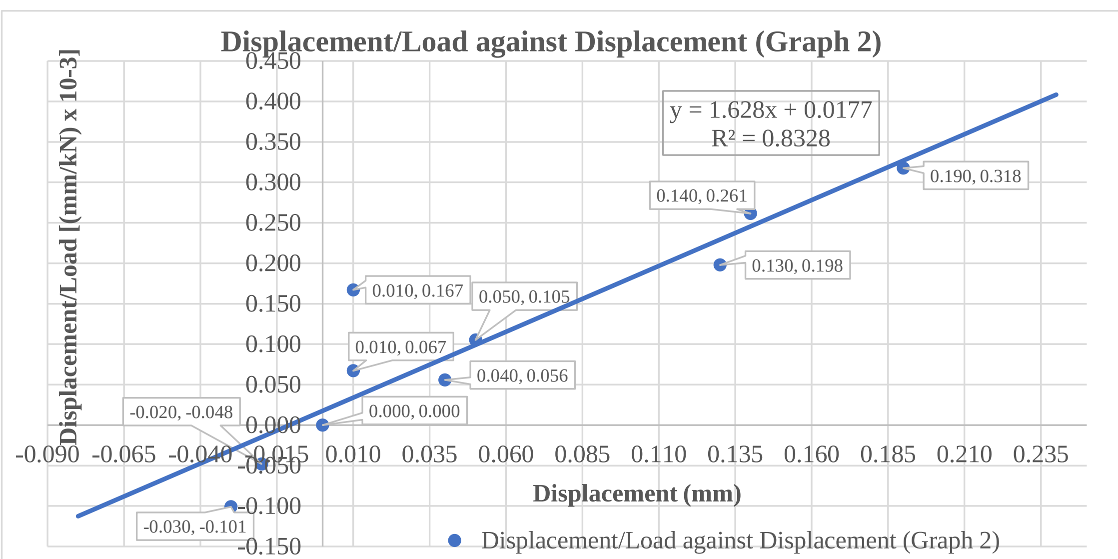
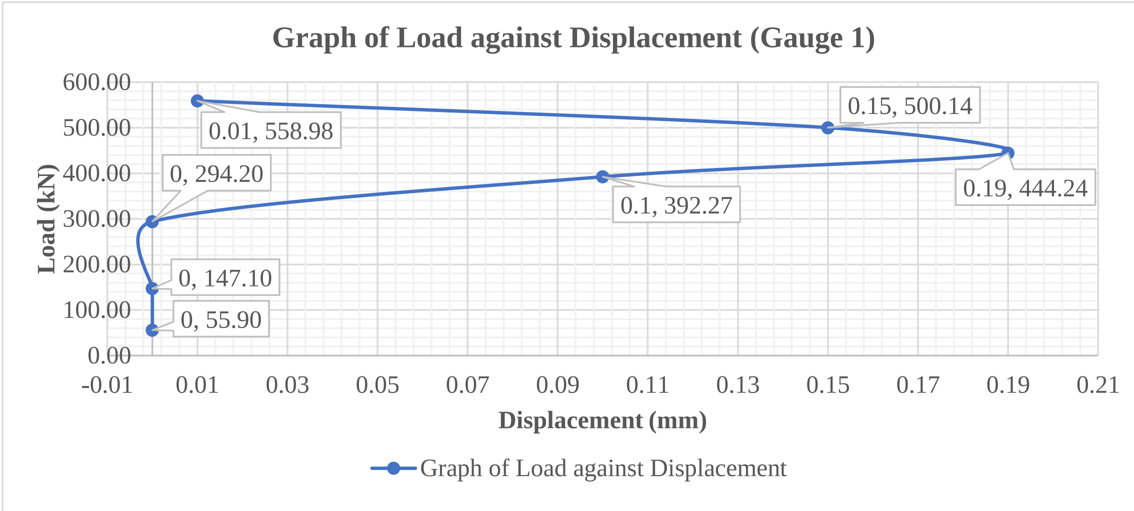
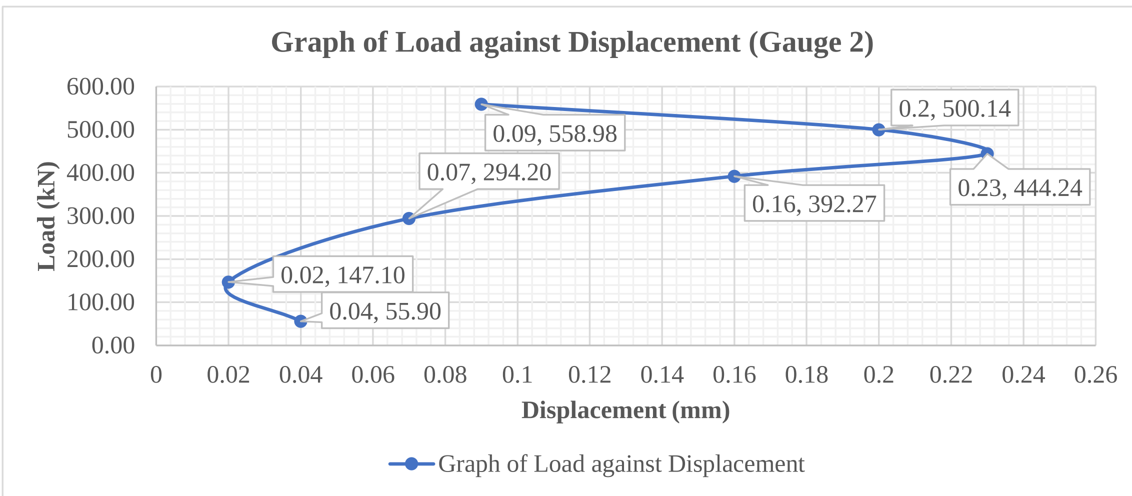


Fig. D-3 (b): AP 108/20- Static Load test Graphs

AP 104/5 UPLIFT LOAD TEST GRAPHS





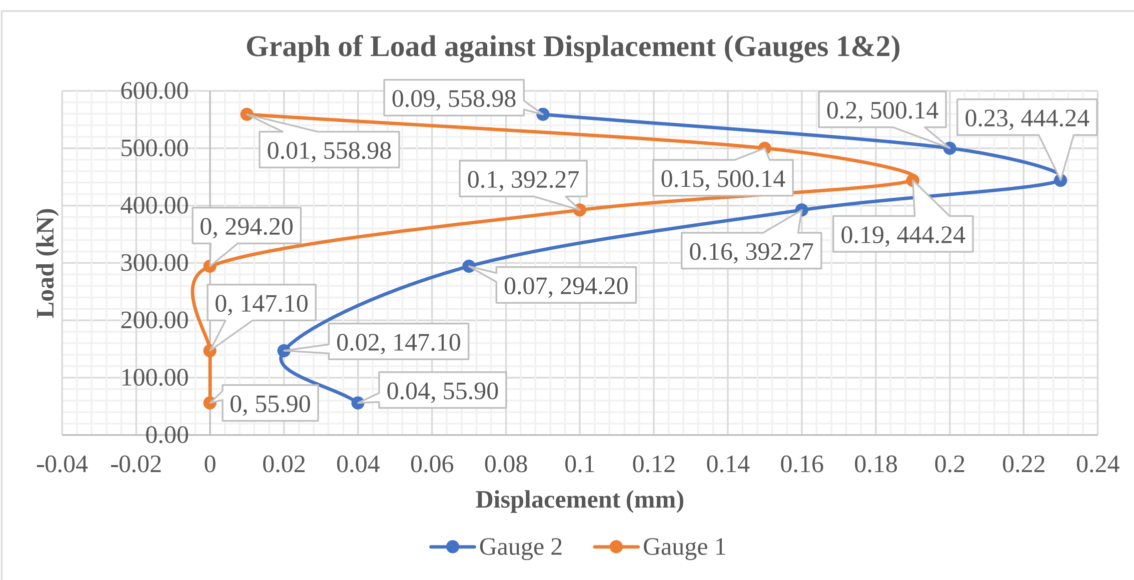
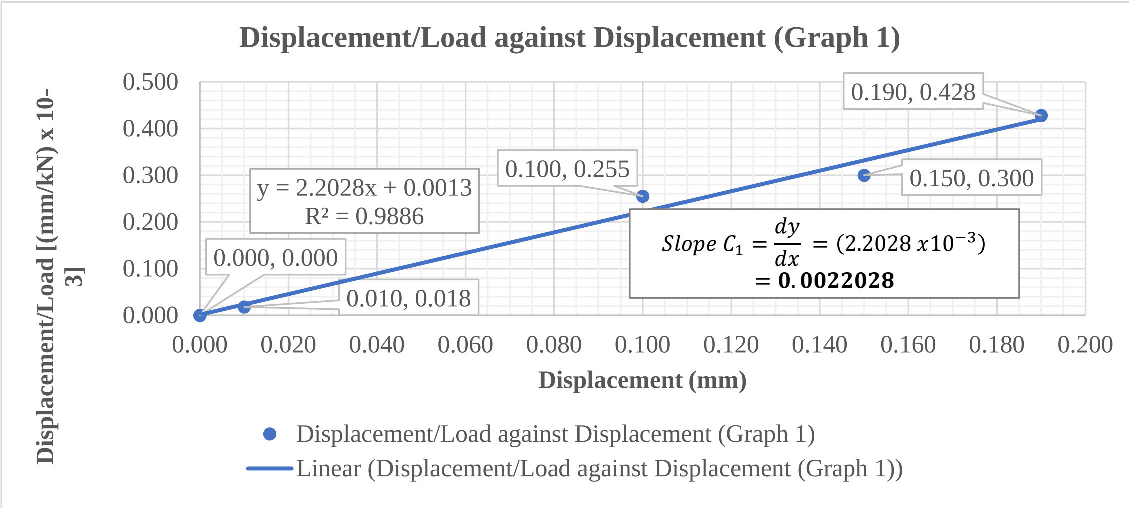
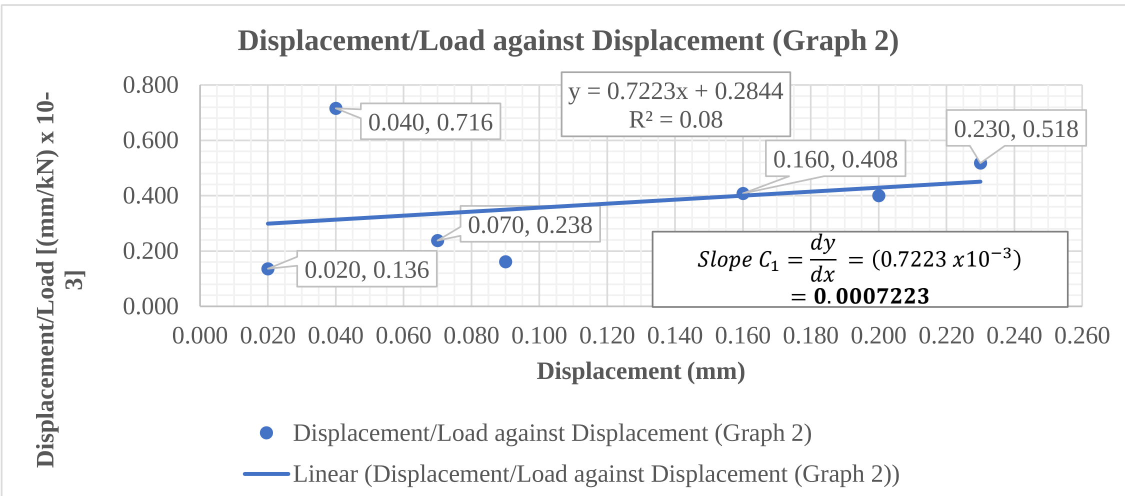


Fig. D-4 (a): AP 104/5- Uplift Load test Graphs

AP 104/5 UPLIFT LOAD TEST GRAPHS





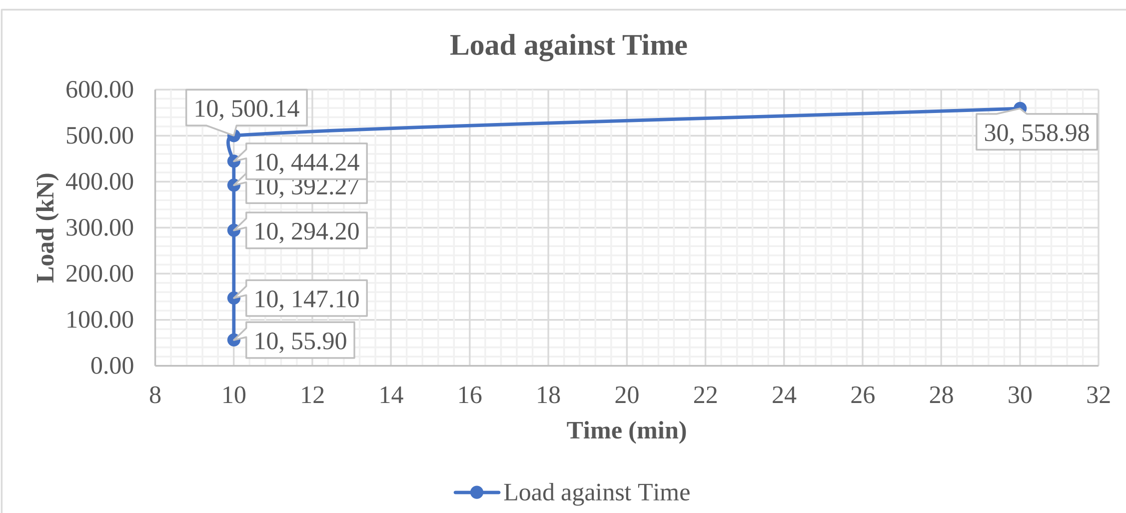


Fig. D-4 (b): AP 104/5- Uplift Load test Graphs

AP 104/5 LATERAL LOAD TEST GRAPHS

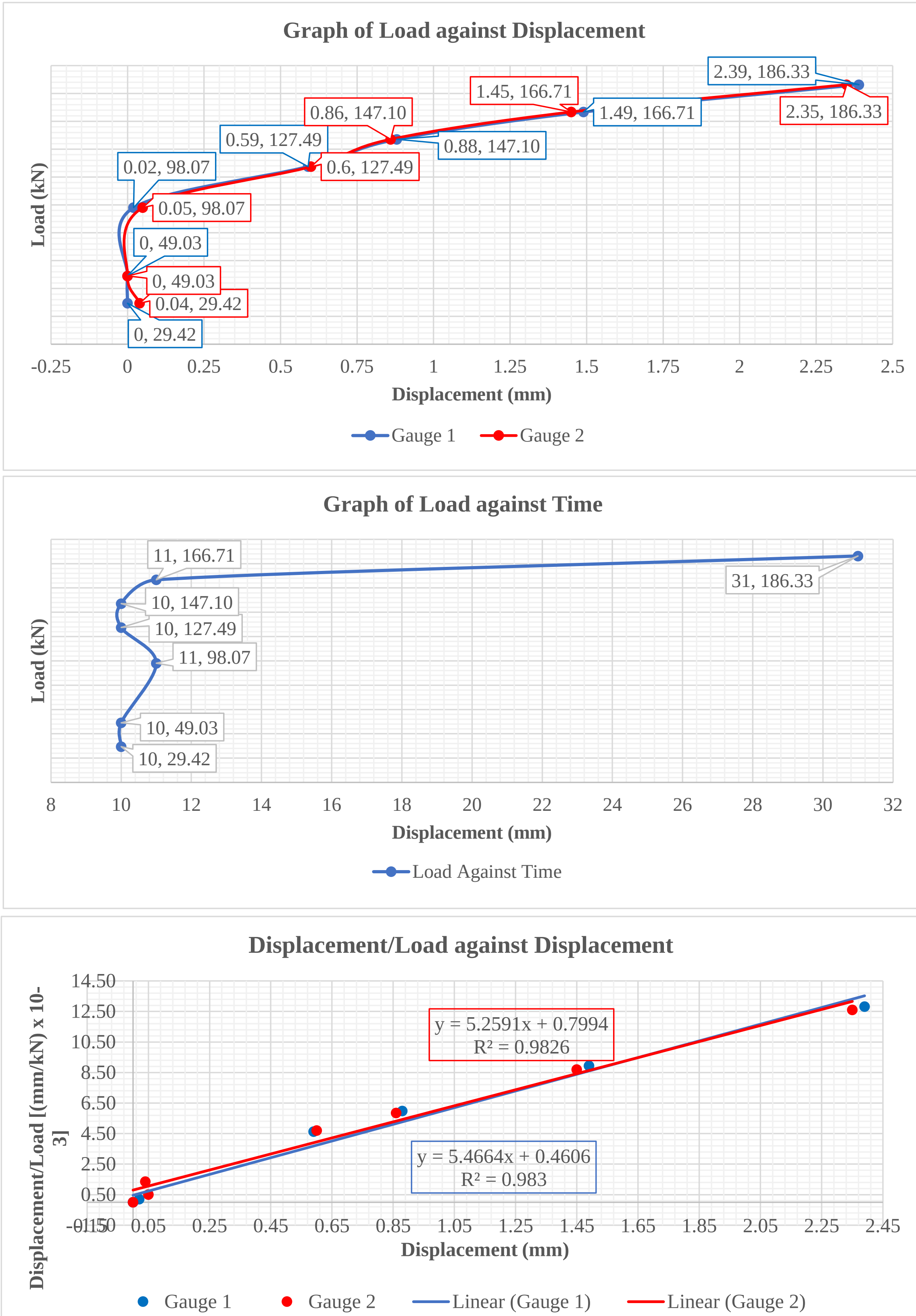
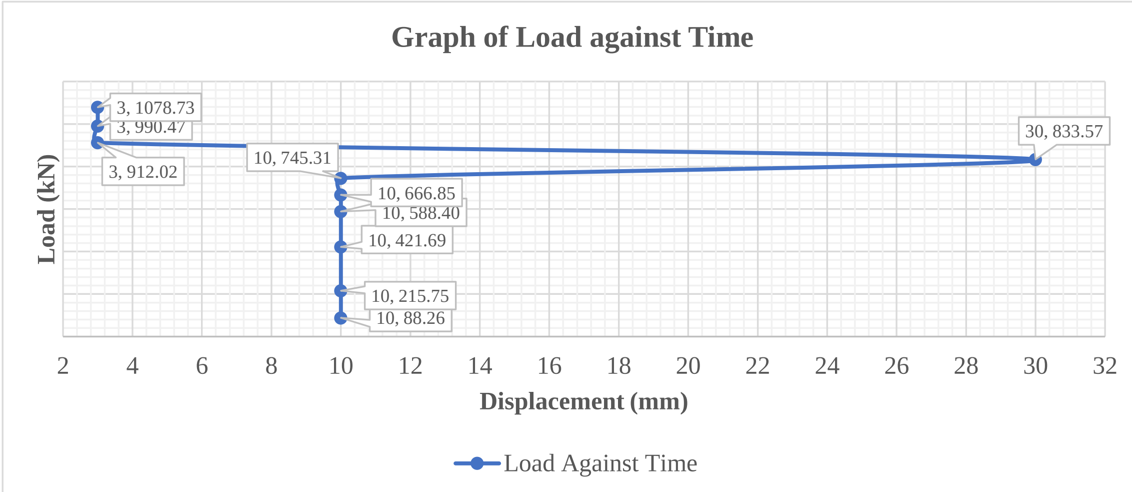
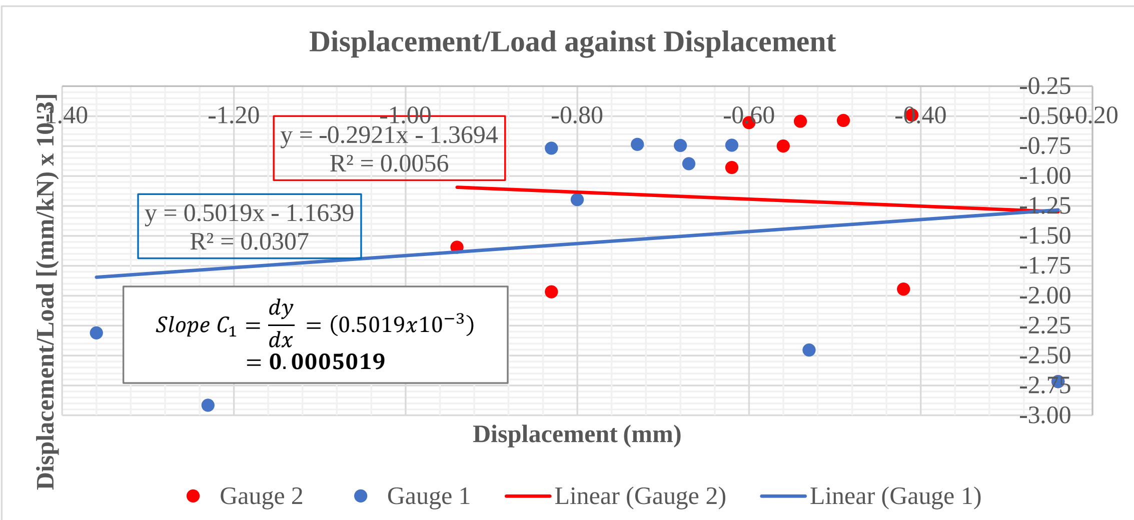


Fig. D-5: AP 104/5- Lateral Load test Graphs

AP 104/5 COMPRESSION LOAD TEST GRAPHS





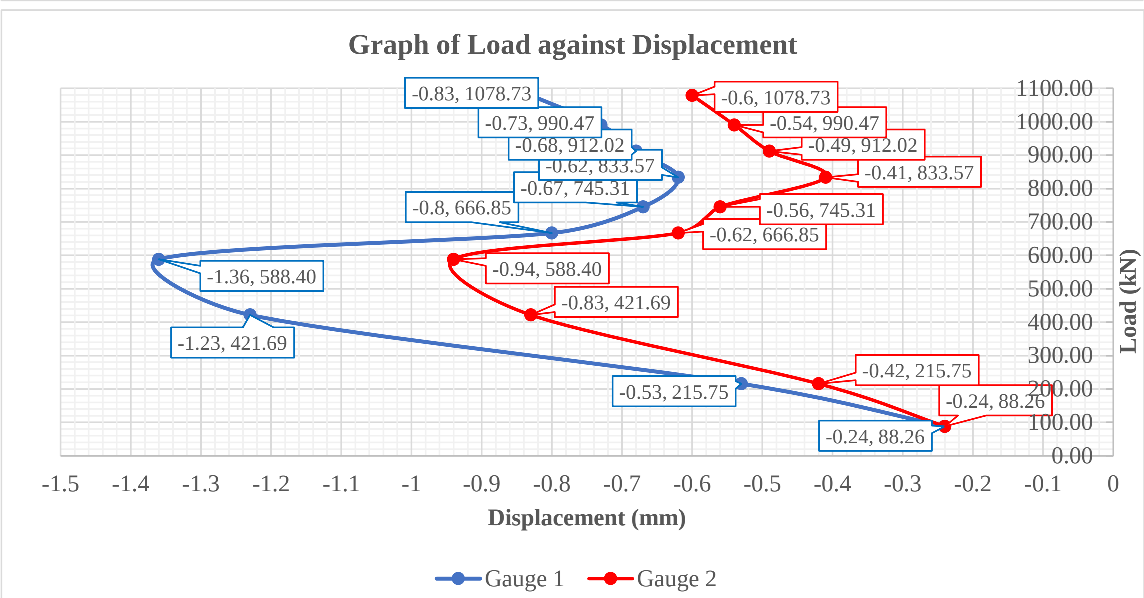


Fig. D-6: AP 104/5- Compression Load test Graphs

Appendix E - Research Time Schedule

Table E-1: Research Time Schedule

S/N	Research Stage	Description	2018					2019												2020											
			8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12	1	2	3	4	5	6	7	8	9	10	11	12
1	Research Conception	Proposal Preparation																													
		Discussion with prospective supervisors																													
2	Research Planning	Proposal reviews by Supervisors																													
		Consultations with Sinohydro, KPTL, etc.																													
		Preliminary site visits to project areas																													
		Subsequent site visits to project areas																													
3	Research Execution	Literature collection & Literature Review																													
		Site desktop analysis works																													
		Surveys & geotechnical investigations																													
		Prescriptive design works																													
		Foundation castings and QA/QC checks																													
		Execution of insitu Static Loading Tests																													GRADUATION CEREMONY
4	Data	Data collection works																													
		Data analysis works																													
5	Report	Preparation of draft reports for review																													
		Final report copy for supervisor's review																													
		Final Report to Graduate School																													
6	Research Publication	Online Research Publication with IJERT																													
7	Research Reviews	Internal Examiners' Reviews																													
		External Examiners' Reviews																													
8	Powerpoint Presentations	Progress presentations																													
		Viva-Voce presentation																													
9	Project Close	Hardcopy Report- binding & submission																													
		Graduation																													

Table E-2: Research Publication Certificate

DOI: 10.13140/RG.2.2.18041.01128 or http://dx.doi.org/10.17577/IJERTV8IS110244

IJERT International Journal of Engineering Research & Technology
ISSN : 2278 - 0181, www.ijert.org
(Published by : ESRSA Publication)

Certificate Of Publication

This is to certify that

Acidri Samuel

Has published a research paper entitled

A Comparative Analysis of Foundations using Prescriptive Design and Static Loading Test Methods

In IJERT, Volume. 8, Issue. 11 , November - 2019

Registration No: IJERTV8IS110244 **Date:** 30-11-2019 **Chief Editor,IJERT**

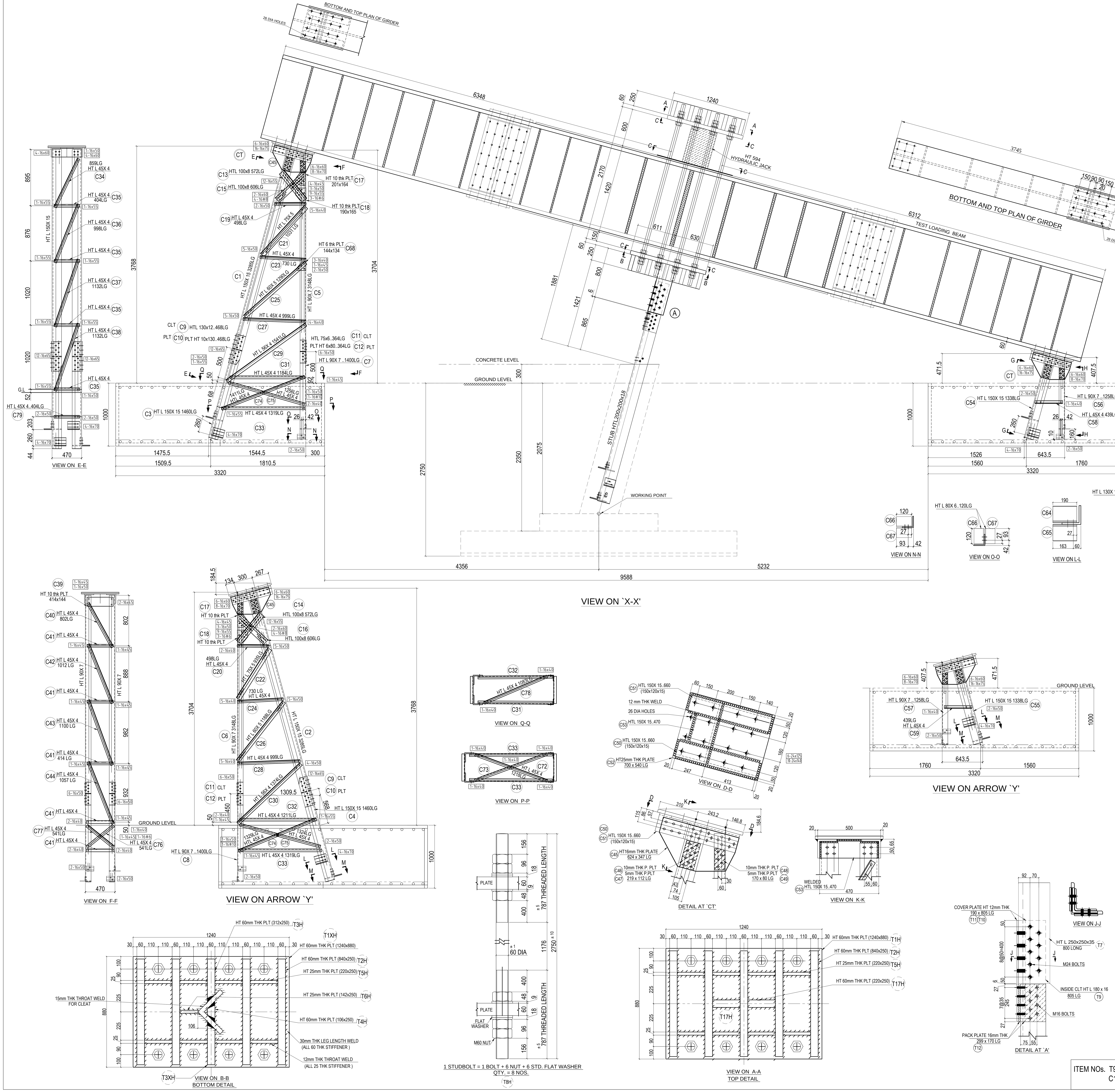
International Journal of Engineering Research & Technology

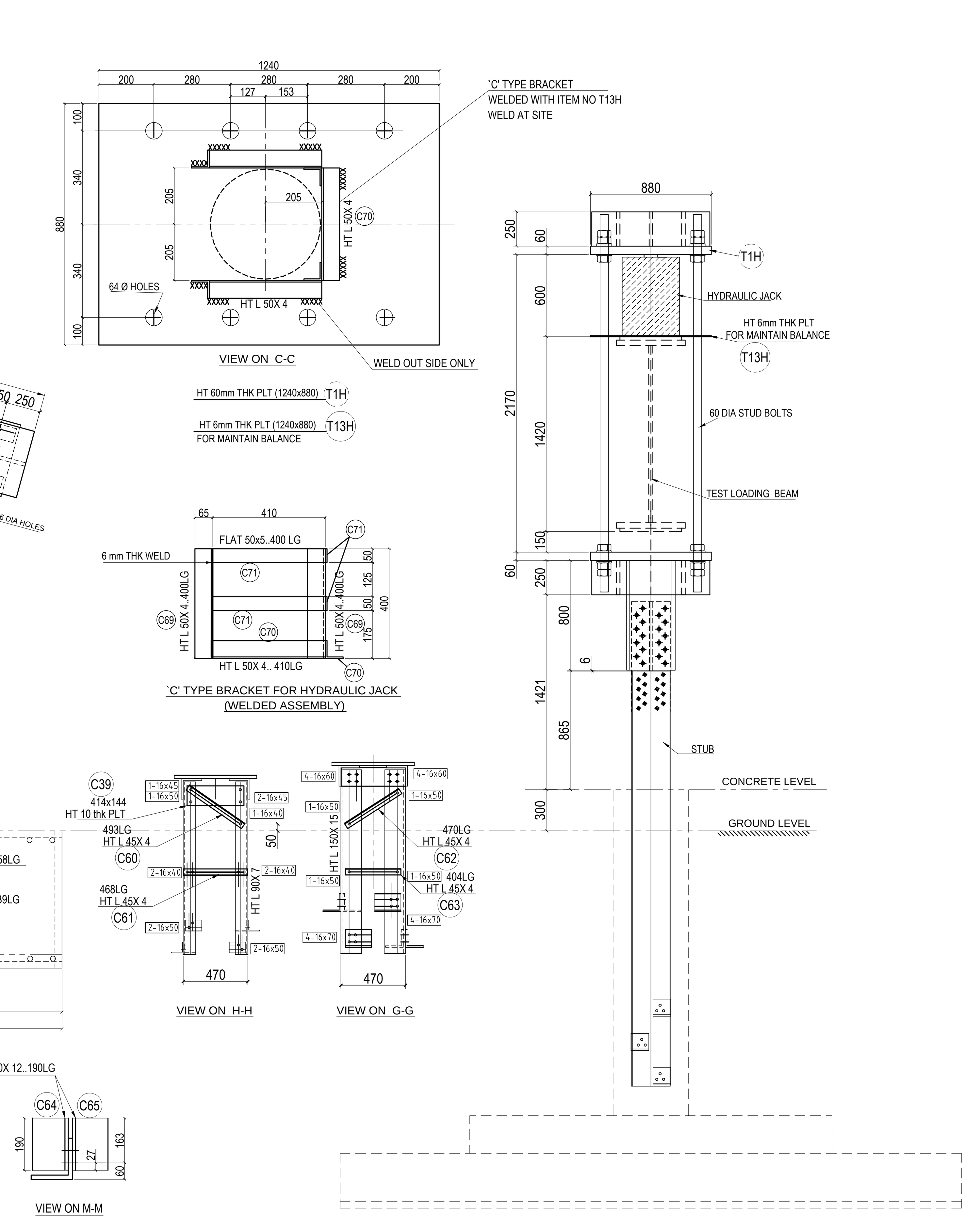
Appendix F - Research Budget

Table F-1: Research Budget

S/No.	Research Cost Breakdown	Qty	Rate	Amount
A	**Preliminary Works**			
A.1	Literature review books (Item)	1	300,000	300,000
A.2	Photocopying of relevant literature (Item)	1	200,000	200,000
A.3	Purchase of stationary material	1	200,000	200,000
A.4	Transportation from Kampala to Karuma and back	6	30,000	180,000
A.5	Field Accommodation and utility payments	3	150,000	450,000
A.6	Phone call communications	12	35,000	420,000
A.7	Internet data services (10 GB MTN data)	12	50,000	600,000
A.8	Meals and refreshment costs (for 4 months in the field)	168	3,000	504,000
A.9	Incidental Expenses	1	100,000	100,000
A.10	Up-keep for field assistants (2 No.) per site	6	70,000	420,000
	Sub-Total 1:			**3,374,000**
B	**Data Collection Works**			
B.1	Site Reconnaissance Visitations	3	15,000	45,000
B.2	Discussions with Stakeholders	6	50,000	300,000
B.3	Geotechnical investigations and/or access to reports	3	450,000	1,350,000
B.4	Conducting and witnessing the Uplift Static Load Test and/or accessing previous test results and reports.	3	750,000	2,250,000
	Sub-Total 2:			**3,945,000**
C	**Report Writing Works**			
C.1	Proposal Report writing and binding works	1	12,500	12,500
C.2	Draft Report Writing for review	2	75,000	150,000
C.3	Final Report Writing and Presentation works	3	150,000	450,000
	Sub-Total 3:			**612,500**
TOTAL				**7,931,500**

Appendix G - Drawings



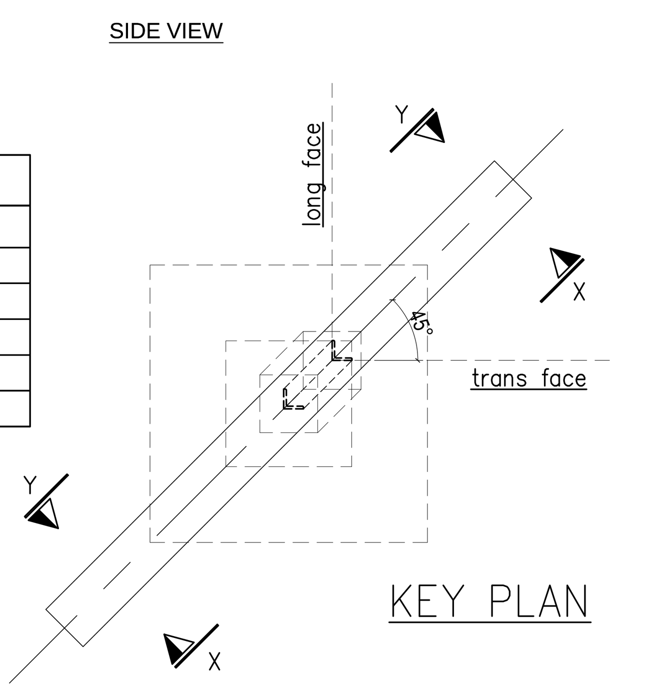


FOR SUPPORT COLUMN

H.R.H. Bolts, Nuts and Washers		
SIZE	M16 QTY.	M24 QTY.
40 mm long bolt	49	-
45 mm long bolt	25	-
50 mm long bolt	96	-
55 mm long bolt	52	-
60 mm long bolt	68	-
65 mm long bolt	48	-
70 mm long bolt	64	-
75 mm long bolt	64	-
125 mm long bolt	-	12
140 mm long bolt	-	20
Spring Washers	466	32
Std. Flat Washers	466	32
6 mm thk Washer	08	-
8 mm thk Washer	08	-
10 mm thk Washer	02	-

FOR STUB JOINT

H.R.H. Bolts, Nuts and Washers		
SIZE	M16 QTY.	M24 QTY.
50 mm long bolt	4	--
95 mm long bolt	16	--
105 mm long bolt	--	18
Spring Washers	20	18
Std. Flat Washers	20	18



NOTES:-

1). All dimensions are in mm.

2). B/N FOR STUB JOINT ;- All bolts and nuts conforming to ISO 898 (Part 1 & 2) with mechanical properties class 8.8 for bolts and class 8 for nuts.

B/N FOR COLUMN ;- All bolts and nuts conforming to ISO 898 (Part 1 & 2) with mechanical properties class 5.6 for bolts and class 5 for nuts.

And Dimensions conforming to ISO 4016 for bolts and ISO 4034 for nuts.

3). All bolts, nuts and washers are to be hot dip galvanized according to ISO 1461.

4). Flat washers as per DIN 126.

5). HT Angle & Plates grade 355 as per BSEN 10025

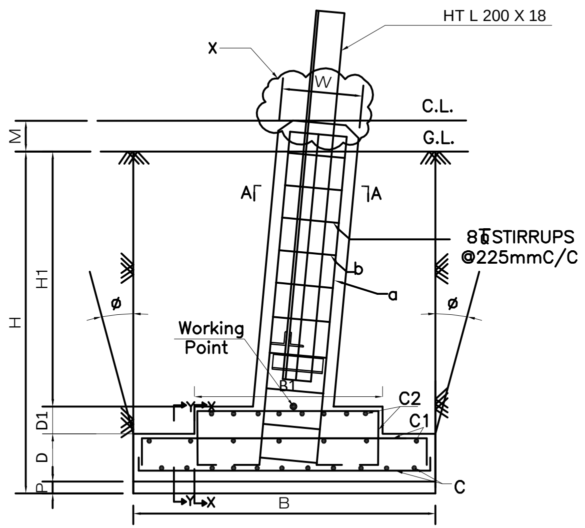
KALPA-TARU POWER TRANSMISSION LIMITED

DRN BY	MAUNISH	18.09.2018
DSG BY	NRA	18.09.2018
CHK BY	PSC	18.09.2018
APPD BY	MBN	18.09.2018

W.O. 295 T.T ST

DETAIL ARRANGEMENT OF FOUNDATION UPLIFT TEST

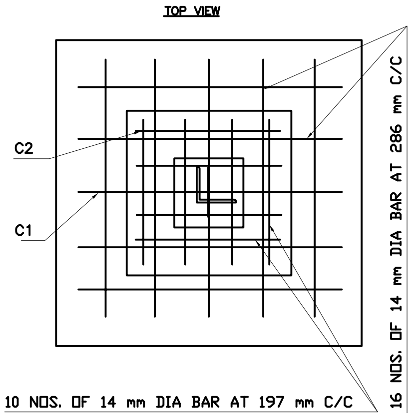
ITEM NOs.	SCALE	DRG. NO. :	SHEET	REV
T9 TO T13 C1 TO C79	NTS	295/FDN/TEST/ST-1		0



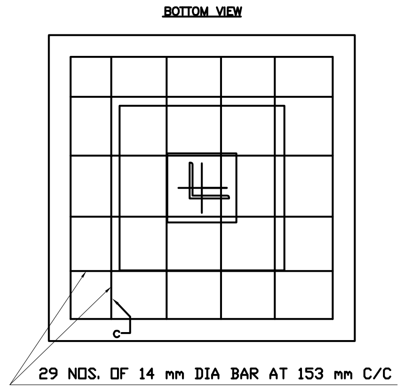
FDN DIM TABLE

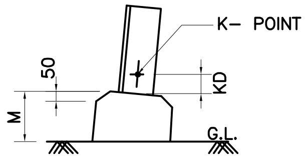
H	2750
B	4390
W	600
M	300
P	50
D	350
D1	275
H1	2075
B1	1880
KD	50

TOP VIEW

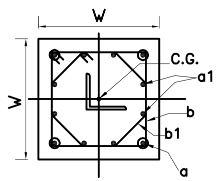


BOTTOM VIEW





DETAIL AT X



SECTION A-A

BAR BENDING DETAIL

MARK	SHAPE & SIZE	DIA	UNIT WT KG/M	LENGTH	QTY / LEG	STEEL FY 500
C2	1780, 445, 150 150, 445	14	1.21	2970	2 X 10	71.88
C1	4290, 170, 170	14	1.21	4630	2 X 16	179.27
C	150, 4290, 150	14	1.21	4590	2 X 29	322.13
a	300, 2900	20	2.469	3200	4	31.60
a1	300, 2900	20	2.469	3200	8	63.21
b	500, 500	8	0.395	2150	13	11.04
b1	225, 180	8	0.395	1770	13	9.09
					TOTAL WEIGHT PER LEG =	688.22kg

QUANTITY/TOWER

CONC.VOL M^3	C 25	34.288
	C 10	3.856
EXCAVATION.VOL m^3		211.992
STEEL KG	FY 500	2753

NOTES:-

1 ALL DIMENSIONS ARE IN mm, UNLESS OTHERWISE SPECIFIED.

2 STEEL USED FOR REINFORCEMENT IS OF GRADE 500. STEEL USED FOR STIRRUPS IS OF GRADE 500.

3 GRADE OF CONCRETE USED IS C25 AS PER SPECIFICATION.

4 MINIMUM COVER TO MAIN REINFORCEMENT SHALL BE 100 mm TO BOTTOM SURFACE & 50 mm TO TOP OF SIDE SURFACE AS PER SPECIFICATION.

5 NO FOUNDATION SHALL REST ON FILLED UP SOIL.

6 NOT MORE THAN 50% BARS SHALL BE LAPPED AT ONE SECTION UNLESS SPECIFIED / SHOWN.

7 ALL HOOKS, BENDS, LAPS, SPLICES & DEVELOPMENT LENGTH SHALL BE AS PER BS: 8110-1985 EXCEPT STATED OTHERWISE.

8 DRAWING NOT TO SCALE

Reference Drawings

Drawing No:	Title:

Approved / Approved with Comments / Not Approved / For Information

Date: Signature / Stamp

Project Consultant

Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: K I P - L - 5 8 0 0 - K - K - D D - C F D - 0 5 2 - 0

	Name:	Date:
Prepared:	V.J.A. / N.R.A.	18.04.2017
Drawn:	P.R.C.	18.04.2017
Checked:	P.S.CHOWDHRY	18.04.2017
Approved:	M.B.NENE	18.04.2017

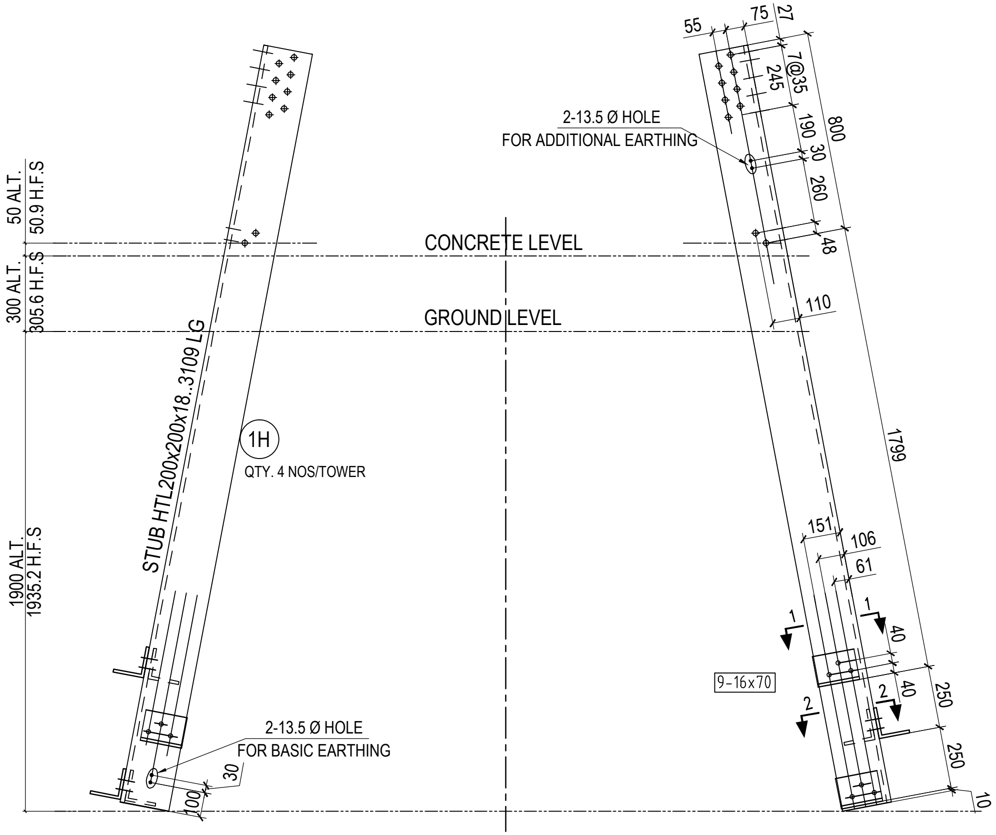
Drawing Detail:

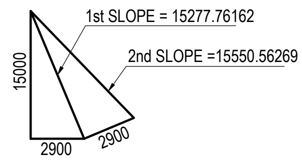
Project: Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line

Title: FOUNDATION SKETCH FOR TT "ST" (0°)-POOR SOIL (DRY)

Revision:			
	D		
	C		
	B		
	A		
	No.	0	18.04.2017

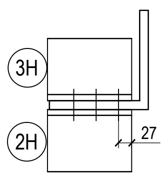
Scale: | Contractor's Drawing No. - - - - - - - 2 9 5 / F D N - 2 S T | Sheet No. 8 | Total Sh. 13



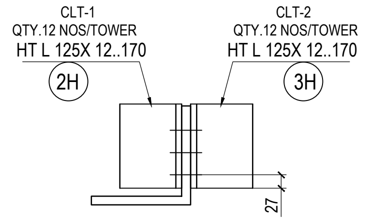


SLOPE DIAGRAM

M16 H.R.H. Bolts, Nuts and Washers with 3% extra	
SIZE	QTY./TOWER
70 mm long bolt	37
Spring Washers	37
Std. Flat Washers	37



VIEW ON 1-1



VIEW ON 2-2

L.D.	M.D.	ITEM NO :-
		1 TO 3

NOTES:-

1). All dimensions are in mm.
2). All holes are 17.5 Ø for 16 Ø bolts noted as ○
3). Mild steel shall conform to BSEN 10025 GR-S 235 JO.
4). High tensile steel shall conform to BSEN 10025 GR-S 355 JO.
5). All steel members are to be hot dip galvanized according to ISO 1461 with uniform zinc coating of min.610 g/m ^{2} & 86µm. thk.
6). Standard spring & flat washers as per DIN 127 (Type B) & DIN 126 respectively to be supplied with each bolt.
7). Min. rolled & cut edge security shall be 22 & 26mm respectively for 16Ø bolts.
8). All item nos are to be prefixed with letter 295DB.
9). All bolts and nuts conforming to ISO 898 (PART 1 & 2) with mechanical properties class 6.8 / 6.
10). All bolts, nuts and washers are to be hot dip galvanized according to ASTM A153.
11). B / N comprises of 1 bolt + 1 nut + 1 spring washer & 1 flat washer.

REFERENCE DRAWINGS	CLIENTS DOCUMENT NO.	REF DRG No.
+0.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-154	295/DB-8
+1.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-156	295/DB-9
+2.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-158	295/DB-10
+3.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-160	295/DB-11
+4.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-162	295/DB-12
STUB SETTING TEMPLATE	KIP-L-5500-K-K-DD-STW-164	295/DB-2

Reference Drawings

Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: ____________ Signature / Stamp ____________ Project Consultant

"Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

"CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP"

Project Drawing: KIP-L-5500-K-K-DD-STW-168-0

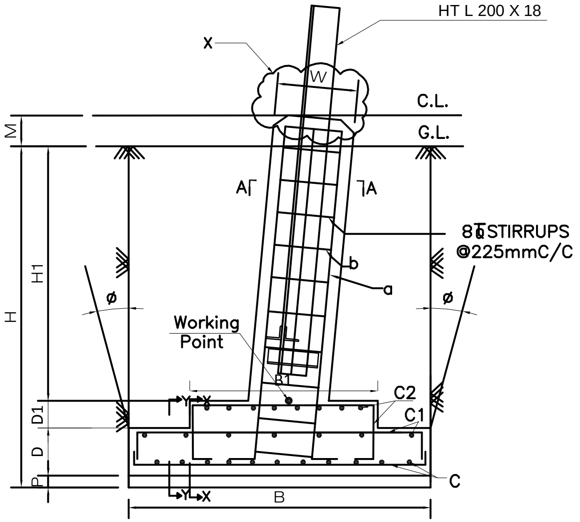
	Name:	Date:
Drawn:	SAGAR	11.04.2017
Checked:	P.C. / NITIN A.	11.04.2017
Reviewed:	P.S.C.	11.04.2017
Approved:	M.B.NENE	11.04.2017

Drawing Detail:
Project:
"Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line"

Revision:		
D		
C		
B		
A		
No.	0	11.04.2017

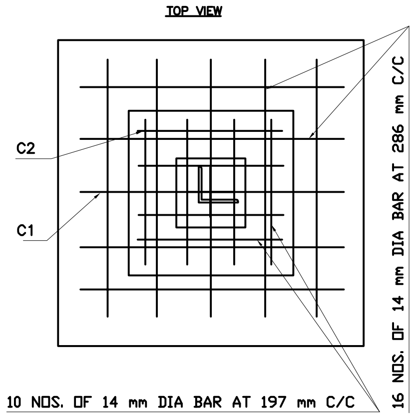
Title:
TOWER TYPE "DB/ST" (2°-30°)
STRUCTURAL DRAWING OF
STUB

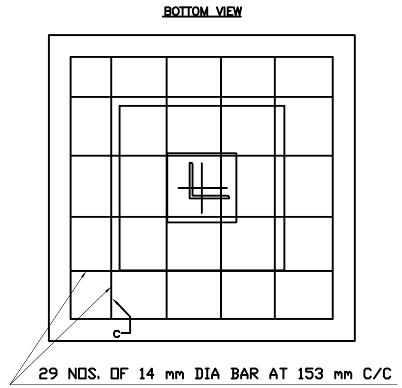
Scale: 1 : 20	"Contractor's Drawing No."	- - - - - - - - 295/DB-01	Sheet No. 1	Total Sh. 1



FDN DIM TABLE

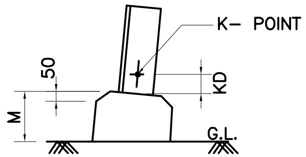
H	2750
B	4390
W	600
M	300
P	50
D	350
D1	275
H1	2075
B1	1880
KD	50



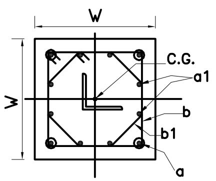


BAR BENDING DETAIL

MARK	SHAPE & SIZE	DIA	UNIT WT KG/M	LENGTH	QTY / LEG	STEEL FY 500
C2	445 1780 445, 150 150	14	1.21	2970	2 X 10	71.88
C1	170 4290 170	14	1.21	4630	2 X 16	179.27
C	150 4290 150	14	1.21	4590	2 X 29	322.13
a	300 2900	20	2.469	3200	4	31.60
a1	300 2900	20	2.469	3200	8	63.21
b	500 500	8	0.395	2150	13	11.04
b1	225 180	8	0.395	1770	13	9.09
					TOTAL WEIGHT PER LEG =	688.22kg



DETAIL AT X



SECTION A-A

QUANTITY/TOWER

CONC.VOL M^3	C 25	34.288
	C 10	3.856
EXCAVATION.VOL m^3		211.992
STEEL KG	FY 500	2753

NOTES:-

1 ALL DIMENSIONS ARE IN mm, UNLESS OTHERWISE SPECIFIED.

2 STEEL USED FOR REINFORCEMENT IS OF GRADE 500. STEEL USED FOR STIRRUPS IS OF GRADE 500.

3 GRADE OF CONCRETE USED IS C25 AS PER SPECIFICATION.

4 MINIMUM COVER TO MAIN REINFORCEMENT SHALL BE 100 mm TO BOTTOM SURFACE & 50 mm TO TOP OF SIDE SURFACE AS PER SPECIFICATION.

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6 NOT MORE THAN 50% BARS SHALL BE LAPPED AT ONE SECTION UNLESS SPECIFIED / SHOWN.

7 ALL HOOKS, BENDS, LAPS, SPLICES & DEVELOPMENT LENGTH SHALL BE AS PER BS: 8110-1985 EXCEPT STATED OTHERWISE.

8 DRAWING NOT TO SCALE

Reference Drawings

Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: ____________ Signature / Stamp ____________

Project Consultant

Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: K I P - L - 5 8 0 0 - K - K - D D - C F D - 0 5 2 - 0

	Name:	Date:
Prepared:	V.J.A. / N.R.A.	18.04.2017
Drawn:	P.R.C.	18.04.2017
Checked:	P.S.CHOWDHRY	18.04.2017
Approved:	M.B.NENE	18.04.2017

Drawing Detail:

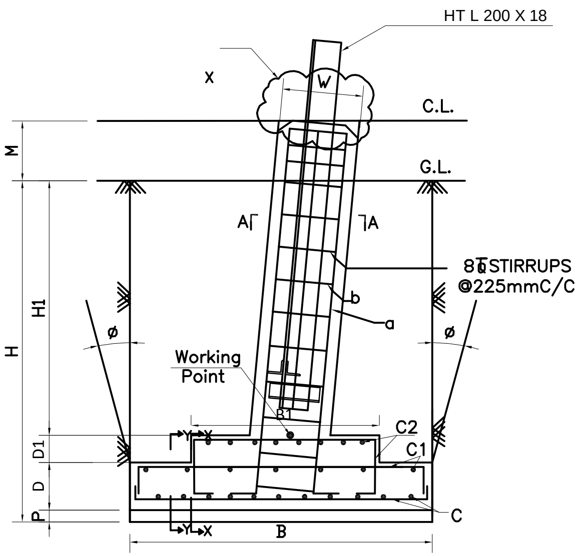
Project:
Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line

Title:
FOUNDATION SKETCH FOR TT "ST" (0°)-POOR SOIL (DRY)

Revision:		
D		
C		
B		
A		
No.	0	18.04.2017

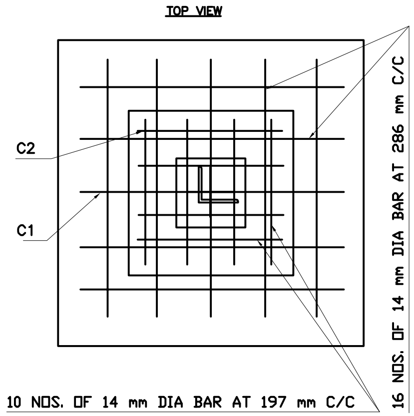
Scale: | Contractor's Drawing No. - - - - - - - 2 9 5 / F D N - 2 S T | Sheet No. 8 | Total Sh. 13

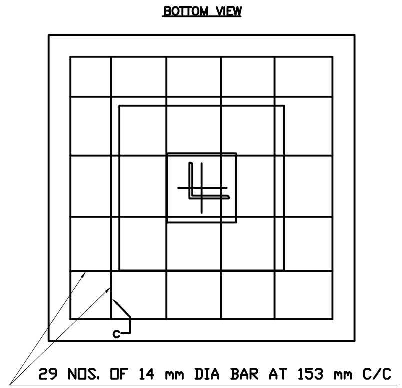
(0.5M RAISED CHIMNEY)



FDN DIM TABLE

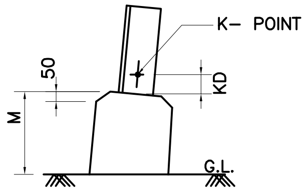
H	2750
B	4390
W	600
M	800
P	50
D	350
D1	275
H1	2075
B1	1880
KD	50



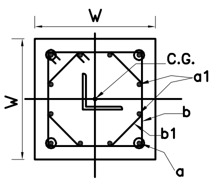


BAR BENDING DETAIL

MARK	SHAPE & SIZE	DIA	UNIT WT KG/M	LENGTH	QTY / LEG	STEEL FY 500
C2	445 1780 445, 150 150	14	1.21	2970	2 X 10	71.88
C1	170 4290 170	14	1.21	4630	2 X 16	179.27
C	150 4290 150	14	1.21	4590	2 X 29	322.13
a	300 3420	20	2.469	3720	4	36.74
a1	300 3420	20	2.469	3720	8	73.48
b	500 500	8	0.395	2150	16	13.59
b1	225 180	8	0.395	1770	16	11.19
					TOTAL WEIGHT PER LEG =	708.28kg



DETAIL AT X



SECTION A-A

NOTES:-

1 ALL DIMENSIONS ARE IN mm, UNLESS OTHERWISE SPECIFIED.

2 STEEL USED FOR REINFORCEMENT IS OF GRADE 500. STEEL USED FOR STIRRUPS IS OF GRADE 500.

3 GRADE OF CONCRETE USED IS C25 AS PER SPECIFICATION.

4 MINIMUM COVER TO MAIN REINFORCEMENT SHALL BE 100 mm TO BOTTOM SURFACE & 50 mm TO TOP OF SIDE SURFACE AS PER SPECIFICATION.

5 NO FOUNDATION SHALL REST ON FILLED UP SOIL.

6 NOT MORE THAN 50% BARS SHALL BE LAPPED AT ONE SECTION UNLESS SPECIFIED / SHOWN.

7 ALL HOOKS, BENDS, LAPS, SPLICES & DEVELOPMENT LENGTH SHALL BE AS PER BS: 8110-1985 EXCEPT STATED OTHERWISE.

8 DRAWING NOT TO SCALE

QUANTITY/TOWER

CONC.VOL M^3	C 25	35.008
	C 10	3.856
EXCAVATION.VOL m^3		211.992
STEEL KG	FY 500	2833

Reference Drawings

Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: Signature / Stamp

Project Consultant

Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: K I P - L - 5 8 0 0 - K - K - D D - C F D - 0 5 2 - 0

	Name:	Date:
Prepared:	V.J.A. / N.R.A.	18.04.2017
Drawn:	P.R.C.	18.04.2017
Checked:	P.S.CHOWDHRY	18.04.2017
Approved:	M.B.NENE	18.04.2017

Drawing Detail:

Project:
Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line

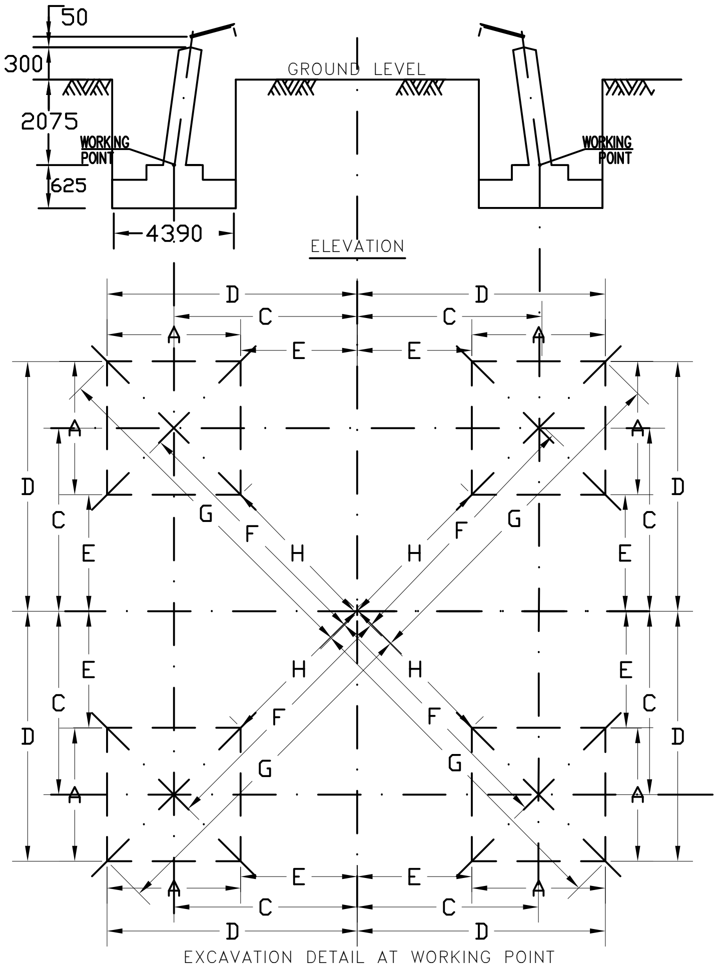
Title:
FOUNDATION SKETCH FOR TT "ST" (0°)-POOR SOIL (DRY)
(0.5M RAISED CHIMNEY)

Revision:	No.		
	D		
	C		
	B		
	A		
	No.	0	18.04.2017

Scale: | Contractor's Drawing No. - - - - - - - 2 9 5 / F D N - 2 S T | Sheet No. 9 | Total Sh. 13

SHEET NO.: 10 OF 13
DESIGN NO: 295/FDN-2ST
TOWER TYPE : TT ST
SOIL TYPE : POOR SOIL (DRY)

TYPICAL EXCAVATION DETAIL

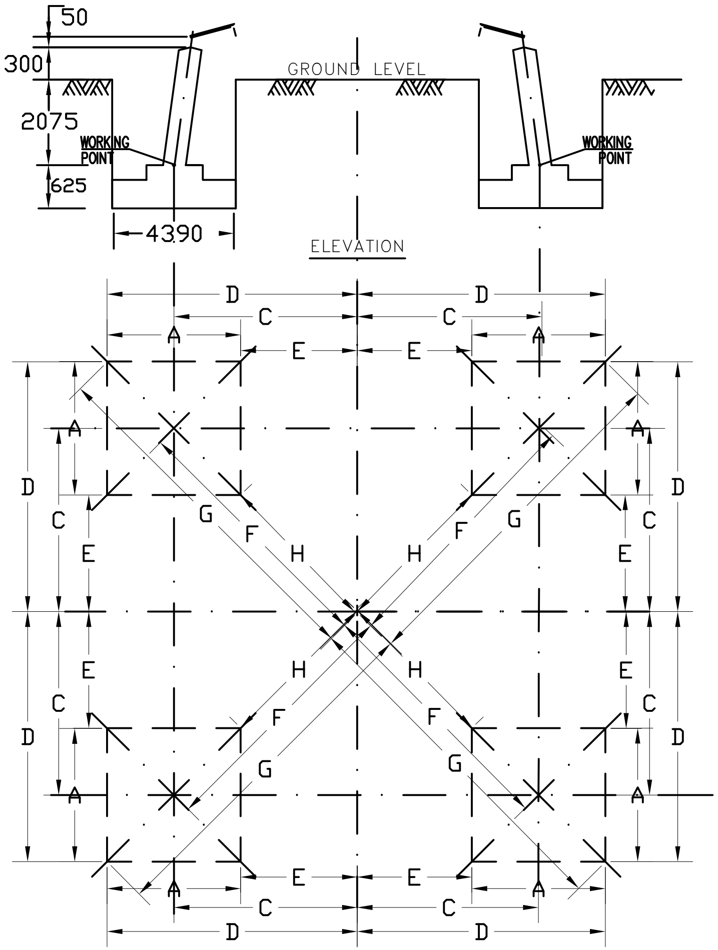


EXCAVATION DIMENSION TABLE

SOIL TYPE	TOWER TYPE	A	C	D	E	F	G	H
POOR SOIL	BTB + 0M LE	4390	6655	8850	4460	9412	12516	6308
POOR SOIL	BTB + 1M LE	4390	6848	9043	4653	9685	12789	6581
POOR SOIL	BTB + 2M LE	4390	7042	9237	4847	9959	13064	6855
POOR SOIL	BTB + 3M LE	4390	7235	9430	5040	10232	13337	7128
POOR SOIL	BTB + 4M LE	4390	7428	9623	5233	10505	13609	7401
POOR SOIL	BTB + 3M BE + 0M LE	4390	7235	9430	5040	10232	13337	7128
POOR SOIL	BTB + 3M BE + 1M LE	4390	7428	9623	5233	10505	13609	7401
POOR SOIL	BTB + 3M BE + 2M LE	4390	7622	9817	5427	10780	13884	7675
POOR SOIL	BTB + 3M BE + 3M LE	4390	7815	10010	5620	11053	14157	7948
POOR SOIL	BTB + 3M BE + 4M LE	4390	8008	10203	5813	11326	14430	8221
POOR SOIL	BTB + 6M BE + 0M LE	4390	7815	10010	5620	11053	14157	7948
POOR SOIL	BTB + 6M BE + 1M LE	4390	8008	10203	5813	11326	14430	8221
POOR SOIL	BTB + 6M BE + 2M LE	4390	8202	10397	6007	11600	14704	8496
POOR SOIL	BTB + 6M BE + 3M LE	4390	8395	10590	6200	11873	14977	8769
POOR SOIL	BTB + 6M BE + 4M LE	4390	8588	10783	6393	12146	15250	9042

SHEET NO.: 11 OF 13
DESIGN NO: 295/FDN-2ST
TOWER TYPE : TT ST
SOIL TYPE : POOR SOIL (DRY)

TYPICAL EXCAVATION DETAIL



EXCAVATION DETAIL AT WORKING POINT

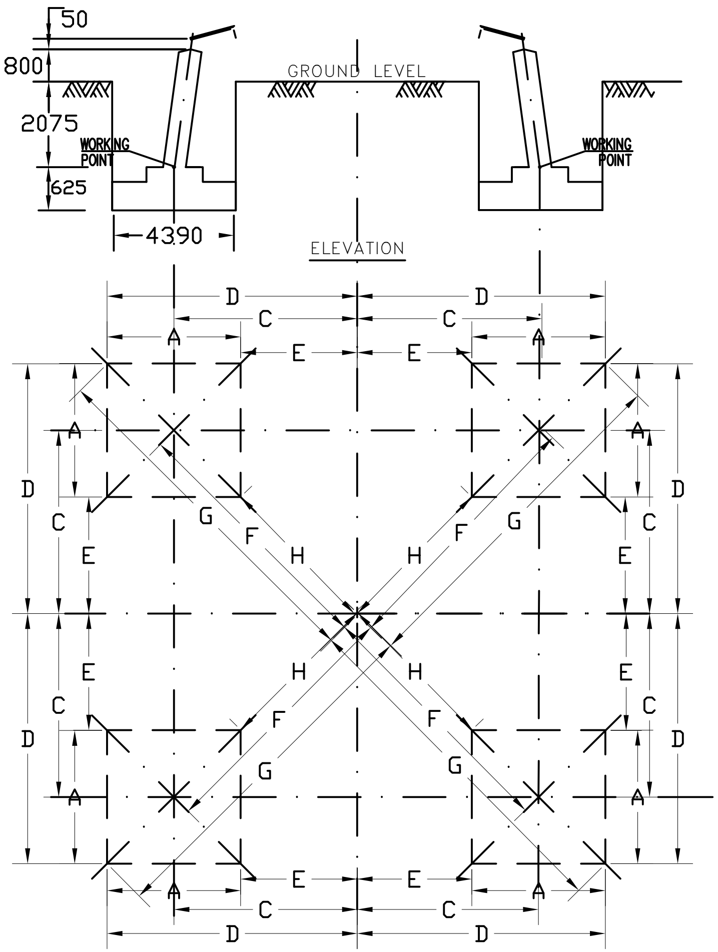
EXCAVATION DIMENSION TABLE

SOIL TYPE	TOWER TYPE	A	C	D	E	F	G	H
POOR SOIL	BTB + 9M BE + 0M LE	4390	8395	10590	6200	11873	14977	8769
POOR SOIL	BTB + 9M BE + 1M LE	4390	8588	10783	6393	12146	15250	9042
POOR SOIL	BTB + 9M BE + 2M LE	4390	8782	10977	6587	12420	15524	9316
POOR SOIL	BTB + 9M BE + 3M LE	4390	8975	11170	6780	12693	15797	9589
POOR SOIL	BTB + 9M BE + 4M LE	4390	9168	11363	6973	12966	16070	9862
POOR SOIL	BTB + 12M BE + 0M LE	4390	8975	11170	6780	12693	15797	9589
POOR SOIL	BTB + 12M BE + 1M LE	4390	9168	11363	6973	12966	16070	9862
POOR SOIL	BTB + 12M BE + 2M LE	4390	9362	11557	7167	13240	16345	10136
POOR SOIL	BTB + 12M BE + 3M LE	4390	9555	11750	7360	13513	16618	10409
POOR SOIL	BTB + 12M BE + 4M LE	4390	9748	11943	7553	13786	16890	10682

B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) = 12484 mm

(0.5M RAISED CHIMNEY)

TYPICAL EXCAVATION DETAIL



EXCAVATION DETAIL AT WORKING POINT

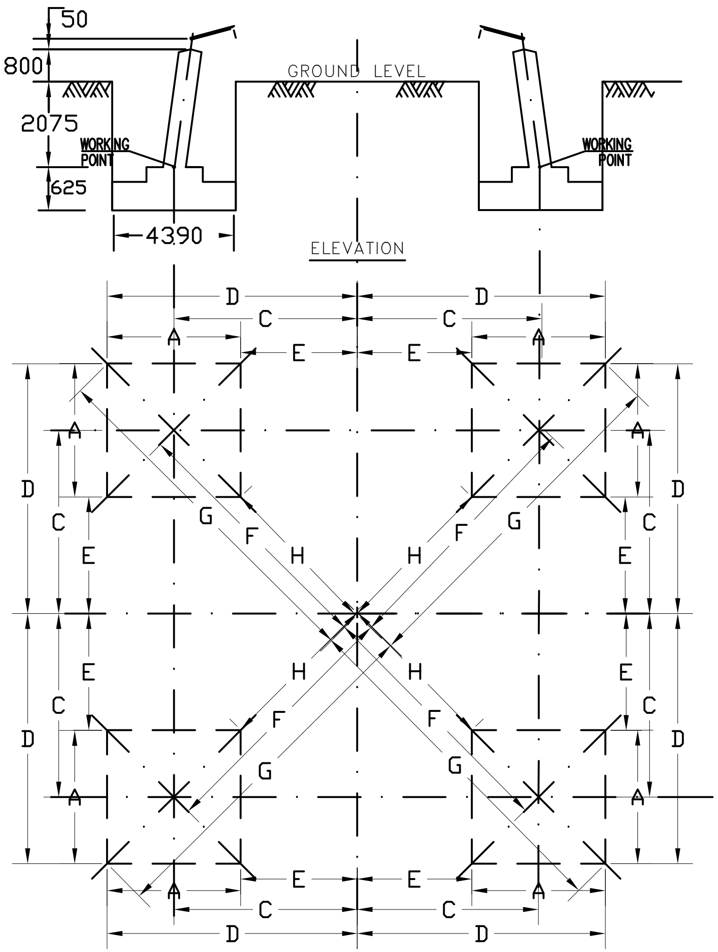
EXCAVATION DIMENSION TABLE

SOIL TYPE	TOWER TYPE	A	C	D	E	F	G	H
POOR SOIL	BTB + 0M LE	4390	6752	8947	4557	9549	12653	6445
POOR SOIL	BTB + 1M LE	4390	6945	9140	4750	9822	12926	6718
POOR SOIL	BTB + 2M LE	4390	7138	9333	4943	10095	13199	6991
POOR SOIL	BTB + 3M LE	4390	7332	9527	5137	10370	13474	7265
POOR SOIL	BTB + 4M LE	4390	7525	9720	5330	10642	13747	7538
POOR SOIL	BTB + 3M BE + 0M LE	4390	7332	9527	5137	10370	13474	7265
POOR SOIL	BTB + 3M BE + 1M LE	4390	7525	9720	5330	10642	13747	7538
POOR SOIL	BTB + 3M BE + 2M LE	4390	7718	9913	5523	10915	14020	7811
POOR SOIL	BTB + 3M BE + 3M LE	4390	7912	10107	5717	11190	14294	8086
POOR SOIL	BTB + 3M BE + 4M LE	4390	8105	10300	5910	11463	14567	8359
POOR SOIL	BTB + 6M BE + 0M LE	4390	7912	10107	5717	11190	14294	8086
POOR SOIL	BTB + 6M BE + 1M LE	4390	8105	10300	5910	11463	14567	8359
POOR SOIL	BTB + 6M BE + 2M LE	4390	8298	10493	6103	11736	14840	8631
POOR SOIL	BTB + 6M BE + 3M LE	4390	8492	10687	6297	12010	15114	8906
POOR SOIL	BTB + 6M BE + 4M LE	4390	8685	10880	6490	12283	15387	9179

(0.5M RAISED CHIMNEY)

SHEET NO.: 13 OF 13
DESIGN NO: 295/FDN-2ST
TOWER TYPE : TT ST
SOIL TYPE : POOR SOIL (DRY)
(0.5m Raised Chimney)

TYPICAL EXCAVATION DETAIL

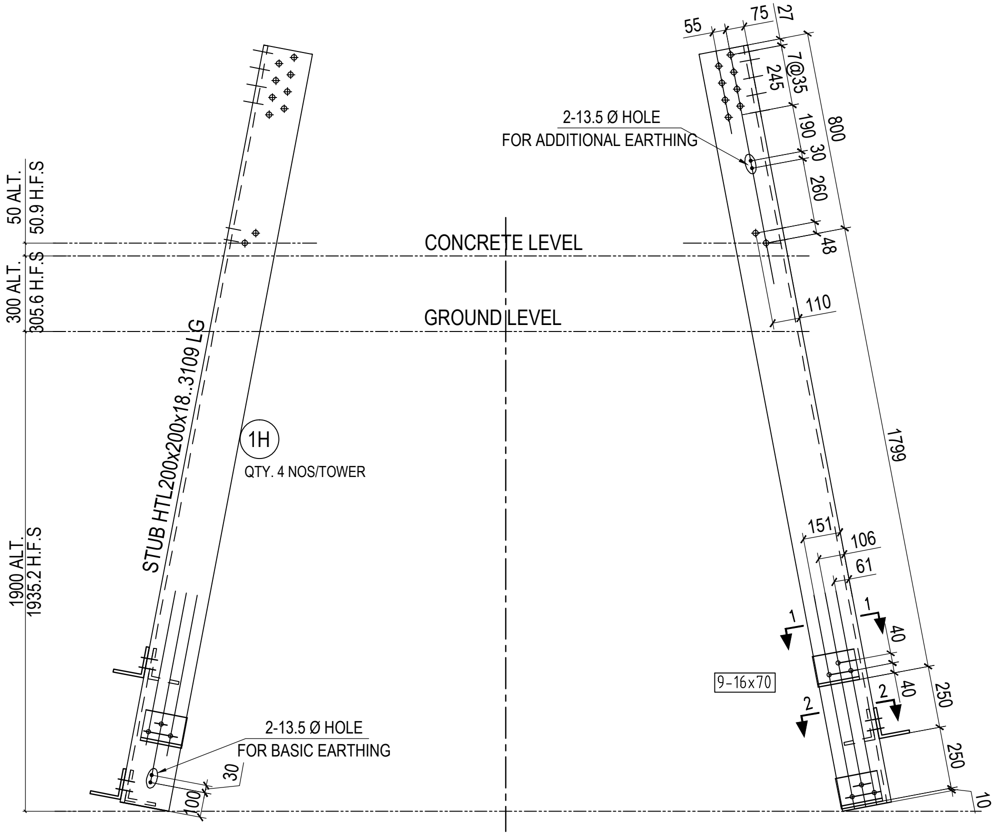


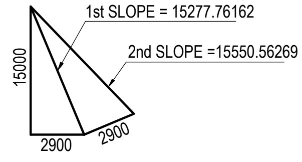
EXCAVATION DETAIL AT WORKING POINT

EXCAVATION DIMENSION TABLE

SOIL TYPE	TOWER TYPE	A	C	D	E	F	G	H
POOR SOIL	BTB + 9M BE + 0M LE	4390	8492	10687	6297	12010	15114	8906
POOR SOIL	BTB + 9M BE + 1M LE	4390	8685	10880	6490	12283	15387	9179
POOR SOIL	BTB + 9M BE + 2M LE	4390	8878	11073	6683	12556	15660	9452
POOR SOIL	BTB + 9M BE + 3M LE	4390	9072	11267	6877	12830	15934	9726
POOR SOIL	BTB + 9M BE + 4M LE	4390	9265	11460	7070	13103	16207	9999
POOR SOIL	BTB + 12M BE + 0M LE	4390	9072	11267	6877	12830	15934	9726
POOR SOIL	BTB + 12M BE + 1M LE	4390	9265	11460	7070	13103	16207	9999
POOR SOIL	BTB + 12M BE + 2M LE	4390	9458	11653	7263	13376	16480	10272
POOR SOIL	BTB + 12M BE + 3M LE	4390	9652	11847	7457	13650	16755	10546
POOR SOIL	BTB + 12M BE + 4M LE	4390	9845	12040	7650	13923	17028	10819

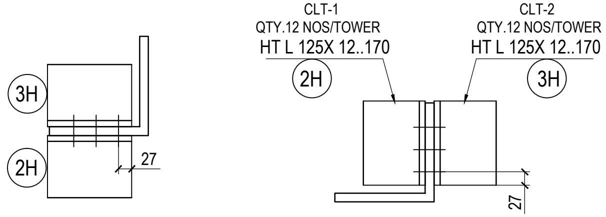
B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) = 12484 mm





SLOPE DIAGRAM

M16 H.R.H. Bolts, Nuts and Washers with 3% extra	
SIZE	QTY./TOWER
70 mm long bolt	37
Spring Washers	37
Std. Flat Washers	37



VIEW ON 1-1

VIEW ON 2-2

L.D.	M.D.	ITEM NO :-
		1 TO 3

NOTES:-

1). All dimensions are in mm.
2). All holes are 17.5 Ø for 16 Ø bolts noted as ○
3). Mild steel shall conform to BSEN 10025 GR-S 235 JO.
4). High tensile steel shall conform to BSEN 10025 GR-S 355 JO.
5). All steel members are to be hot dip galvanized according to ISO 1461 with uniform zinc coating of min.610 g/m^2 & 86µm. thk.
6). Standard spring & flat washers as per DIN 127 (Type B) & DIN 126 respectively to be supplied with each bolt.
7). Min. rolled & cut edge security shall be 22 & 26mm respectively for 16Ø bolts.
8). All item nos are to be prefixed with letter 295DB.
9). All bolts and nuts conforming to ISO 898 (PART 1 & 2) with mechanical properties class 6.8 / 6.
10). All bolts, nuts and washers are to be hot dip galvanized according to ASTM A153.
11). B / N comprises of 1 bolt + 1 nut + 1 spring washer & 1 flat washer.

REFERENCE DRAWINGS	CLIENTS DOCUMENT NO.	REF DRG No.
+0.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-154	295/DB-8
+1.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-156	295/DB-9
+2.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-158	295/DB-10
+3.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-160	295/DB-11
+4.0M LEG EXTENSION	KIP-L-5500-K-K-DD-STW-162	295/DB-12
STUB SETTING TEMPLATE	KIP-L-5500-K-K-DD-STW-164	295/DB-2

Reference Drawings

Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: ____________ Signature / Stamp ____________ Project Consultant

"Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

"CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP"

Project Drawing: K I P - L - 5 5 0 0 - K - K - D D - S T W - 1 6 8 - 0

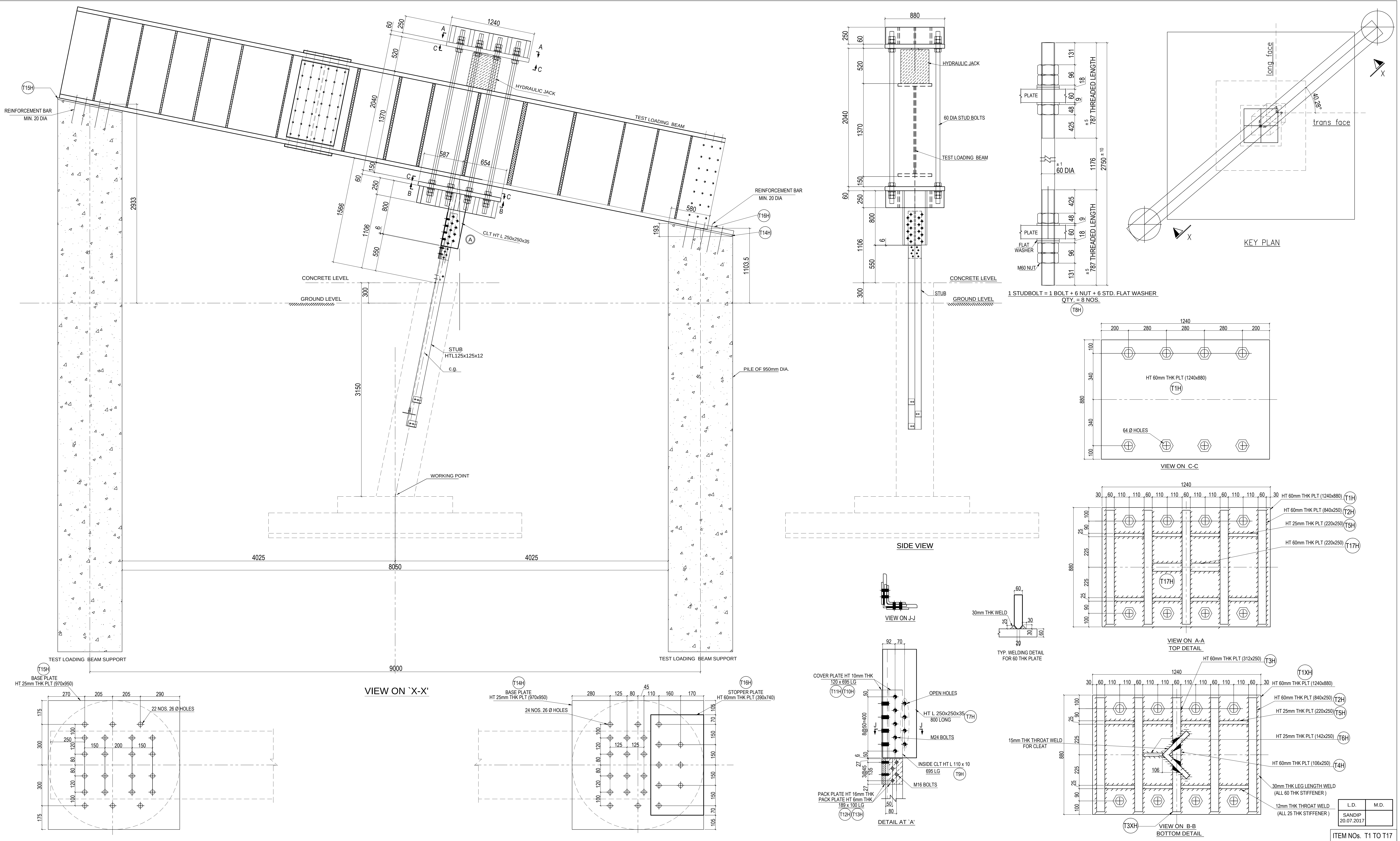
	Name:	Date:
Drawn:	SAGAR	11.04.2017
Checked:	P.C. / NITIN A.	11.04.2017
Reviewed:	P.S.C.	11.04.2017
Approved:	M.B.NENE	11.04.2017

Drawing Detail:
Project:
"Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line"

Revision:		
D		
C		
B		
A		
No.	0	11.04.2017

Title:
TOWER TYPE "DB/ST" (2°-30°)
STRUCTURAL DRAWING OF
STUB

Scale: 1 : 20	"Contractor's Drawing No."	- - - - - - - - 2 9 5 / D B - 0 1	Sheet No. 1	Total Sh. 1



L.D.	M.D.
SANDIP 20.07.2017	

ITEM NOs. T1 TO T17

NOTES:-

1). All dimensions are in mm.
2). All bolts and nuts conforming to ISO 898 (Part 1 & 2) with mechanical properties class 8.8 for bolts and class 8 for nuts. and Dimensions conforming to ISO 4016 for bolts and ISO 4034 for nuts.
3). All bolts, nuts and washers are to be hot dip galvanized according to ISO 1461.
4). Flat washers as per DIN 126.
5). HT Plates grade 355 as per BSEN 10025

FOR STUB JOINT

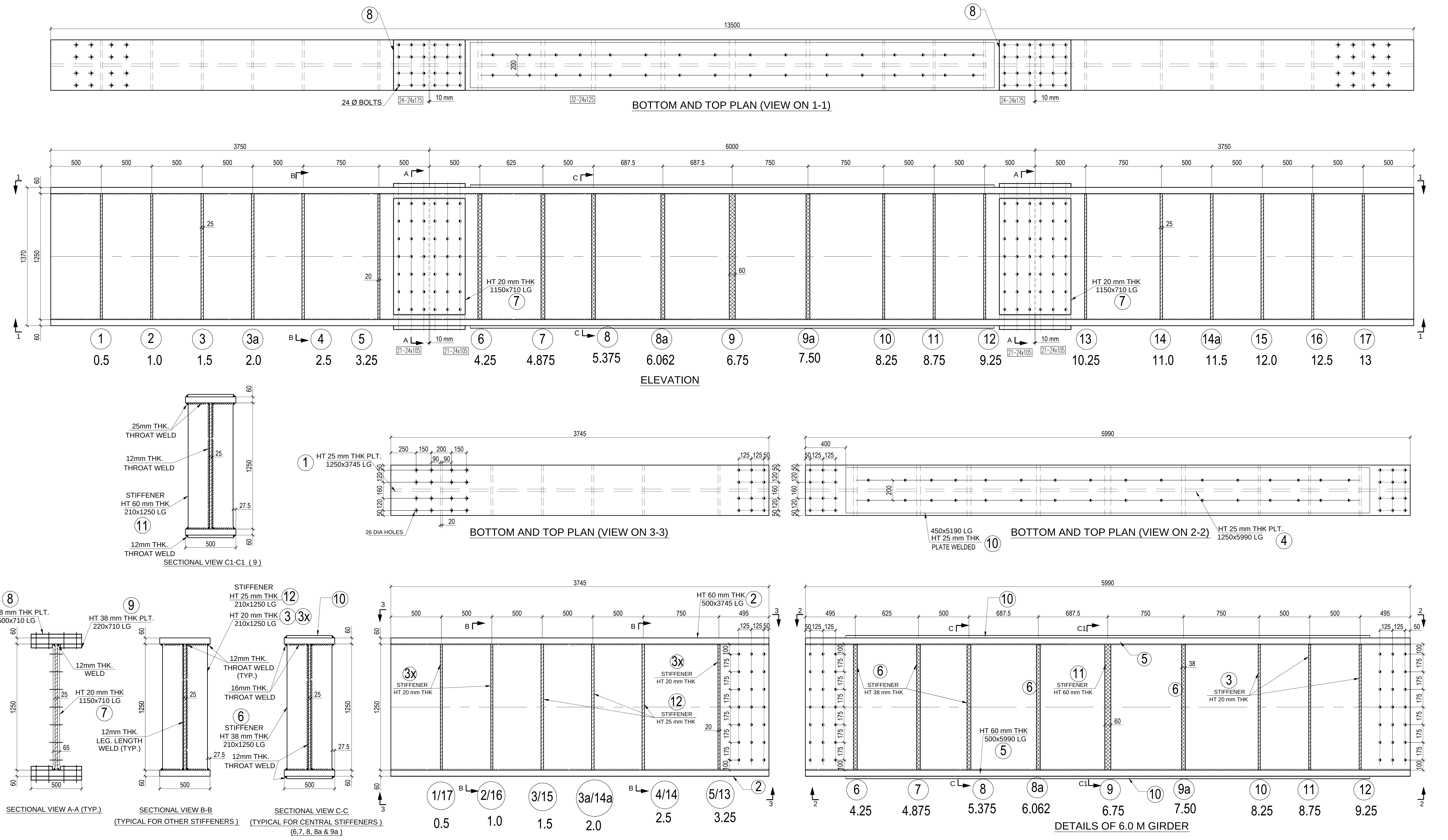
M16 & M24 H.R.H. Bolts, Nuts and Washers		
SIZE	M16 QTY.	M24 QTY.
40 mm long bolt	4	--
85 mm long bolt	8	--
95 mm long bolt	--	9
Spring Washers	12	9
Std. Flat Washers	12	9

KALPA-TARU® POWER TRANSMISSION LIMITED

DRN BY	VBG	17.07.2017
DSG BY	NRA	17.07.2017
CHK BY	PSC	17.07.2017
APPD BY	MBN	17.07.2017

W.O. 295 T.T DA
DETAIL ARRANGEMENT OF FOUNDATION UPLIFT TEST

SCALE	DRG. NO. :	SHEET	REV
NTS	295/FDN/TEST-2		0



GIRDER 13.5 M.
List of 24 Ø H.R.H Bolts,Nuts and Washers
FOR 3.75 M. Long x 2 no. + 6.0 M. Long x 1 no.

SIZE	QTY.
105 mm long bolt	84
125 mm long bolt	64
175 mm long bolt	96
Spring Washers	252
Std. Flat Washers	252

GIRDER 9.75 M.
List of 24 Ø H.R.H Bolts,Nuts and Washers
FOR 3.75 M. Long x 1 no. + 6.0 M. Long x 1 no.

SIZE	QTY.
105 mm long bolt	42
125 mm long bolt	64
175 mm long bolt	48
Spring Washers	162
Std. Flat Washers	162

L.D.	M.D.
SANDIP 06.07.2017 20.07.2017	

SYMBOLS	
◑	26Ø HOLES

MARK No :- 1 TO 12

NOTES:-

1). All dimensions are in mm.
2). All bolts and nuts conforming to ISO 898 (Part 1 & 2) with mechanical properties class 8.8 for bolts and class 8 for nuts. and Dimensions conforming to ISO 4016 for bolts and ISO 4034 for nuts. & threaded length for bolt = 35mm.
3). All bolts, nuts and washers are to be hot dip galvanized according to ISO 1461.
4). Standard spring & flat washers as per DIN 127 (Type B) / DIN 126 to be supplied with each bolt.
5). HT Plates grade 355 as per BSEN 10025
6). STAMPING 295 GIRDERXXXH

KALPA-TARU POWER TRANSMISSION LIMITED

DRN BY	SUDHIR	05.07.2017
DSG BY	PSC	05.07.2017
CHK BY	VBG	05.07.2017
APPD BY	M.B.NENE	05.07.2017

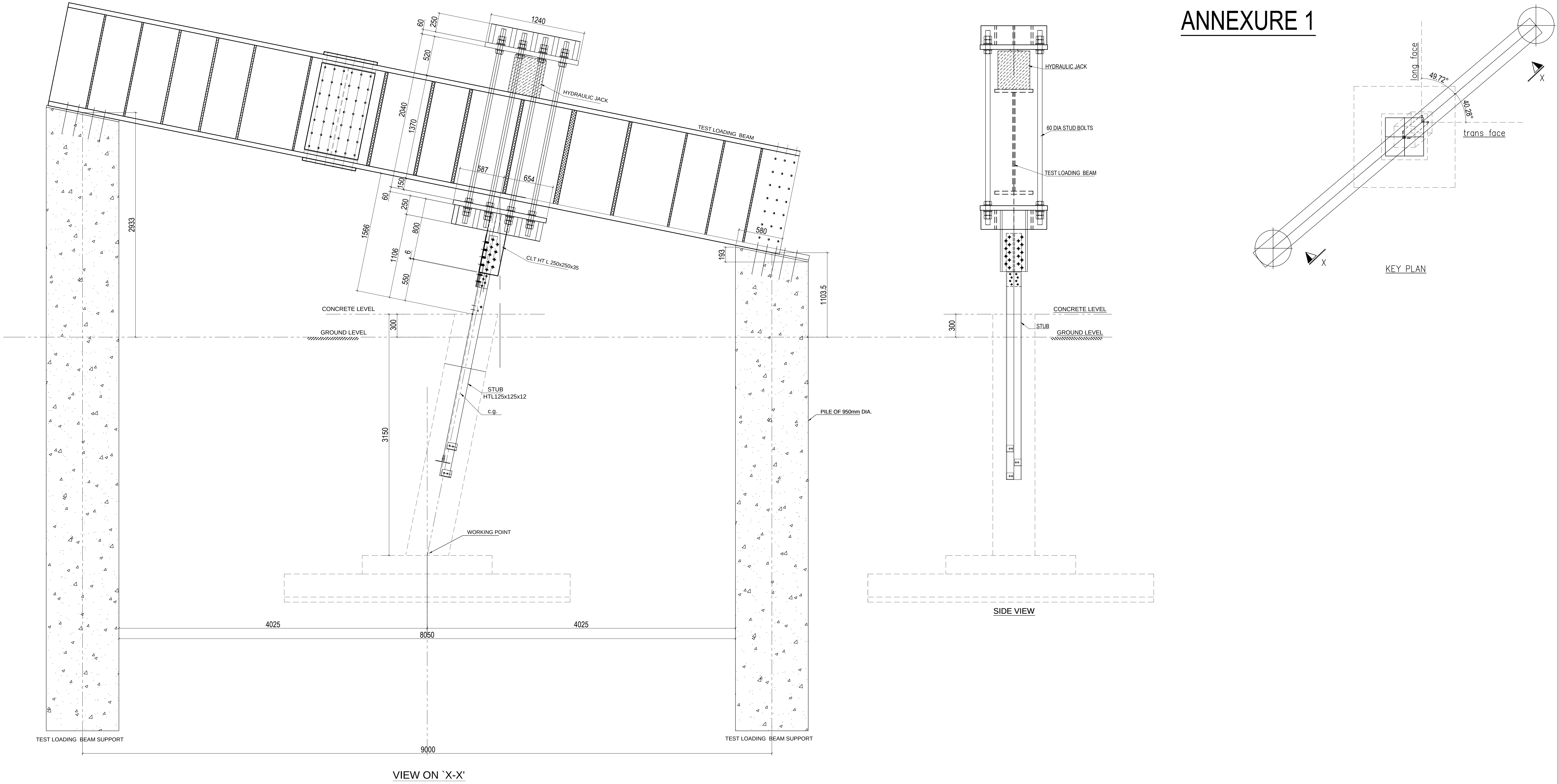
W.O. 295

STRUCTURAL DRG. OF
TEST LOADING ARRANGEMENT PLATE GIRDER 13.5M LONG
3.75 M. Long x 2 no. + 6.0 M. Long x 1 no.
FOR FOUNDATION UPLIFT TEST

SCALE	DRG. NO. :	SHEET	REV
1:25	GIRDER	1 of 1	0

A2

ANNEXURE 1



NOTES:-

1). All dimensions are in mm.

2). As per IEC 1773, Foundation Testing Apparatus are Designed for 1.5 times the Design Load

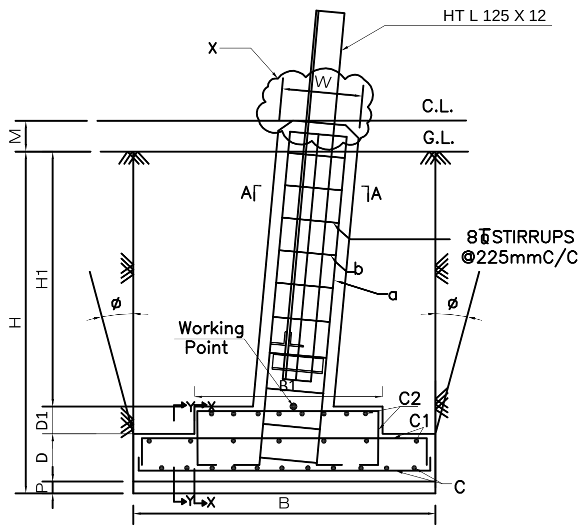
KALPA-TARU POWER TRANSMISSION LIMITED

DRN BY	VBG	17.07.2017
DSG BY	NRA	17.07.2017
CHK BY	PSC	17.07.2017
APPD BY	MBN	17.07.2017

W.O. 295 T.T DA

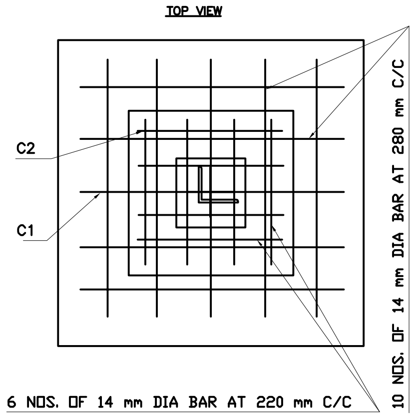
DETAIL ARRANGEMENT OF FOUNDATION UPLIFT TEST

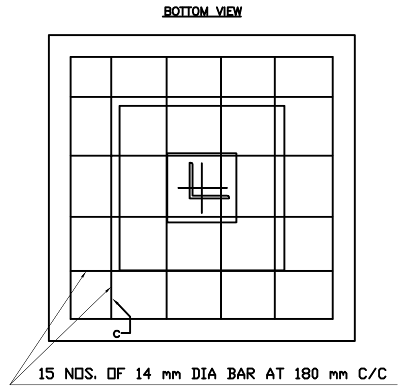
SCALE	DRG. NO. :	SHEET	REV
NTS	295/FDN/TEST-1		0



FDN DIM TABLE

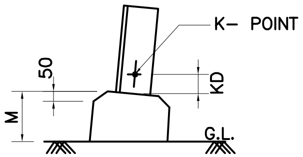
H	3500
B	2640
W	550
M	300
P	50
D	350
D1	250
H1	2850
B1	1200
KD	50



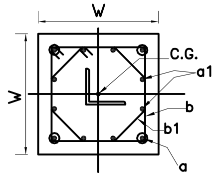


BAR BENDING DETAIL

MARK	SHAPE & SIZE	DIA	UNIT WT KG/M	LENGTH	QTY / LEG	STEEL FY 500
C2	422 1100 150 150 422	14	1.21	2244	2 X 6	32.58
C1	170 2540 170	14	1.21	2880	2 X 10	69.70
C	150 2540 150	14	1.21	2840	2 X 15	103.09
a	300 3625	20	2.469	3925	4	38.76
a1	300 3625	16	1.578	3925	8	49.55
b	450 450	8	0.395	1950	17	13.09
b1	205 160	8	0.395	1610	17	10.81
					TOTAL WEIGHT PER LEG =	317.58kg



DETAIL AT X



SECTION A-A

NOTES:-

1 ALL DIMENSIONS ARE IN mm, UNLESS OTHERWISE SPECIFIED.

2 STEEL USED FOR REINFORCEMENT IS OF GRADE 500. STEEL USED FOR STIRRUPS IS OF GRADE 500.

3 GRADE OF CONCRETE USED IS C25 AS PER SPECIFICATION.

4 MINIMUM COVER TO MAIN REINFORCEMENT SHALL BE 100 mm TO BOTTOM SURFACE & 50 mm TO TOP OF SIDE SURFACE AS PER SPECIFICATION.

5 NO FOUNDATION SHALL REST ON FILLED UP SOIL.

6 NOT MORE THAN 50% BARS SHALL BE LAPPED AT ONE SECTION UNLESS SPECIFIED / SHOWN.

7 ALL HOOKS, BENDS, LAPS, SPLICES & DEVELOPMENT LENGTH SHALL BE AS PER BS: 8110-1985 EXCEPT STATED OTHERWISE.

8 DRAWING NOT TO SCALE

QUANTITY/TOWER

CONC.VOL M^3	C 25	15.009
	C 10	1.394
EXCAVATION.VOL m^3		97.575
STEEL KG	FY 500	1271

Reference Drawings

Drawing No:	Title:

Approved / Approved with Comments / Not Approved / For Information

Date: Signature / Stamp

Project Consultant

Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: K I P - L - 5 8 0 0 - K - K - D D - C F D - 0 0 1 - B

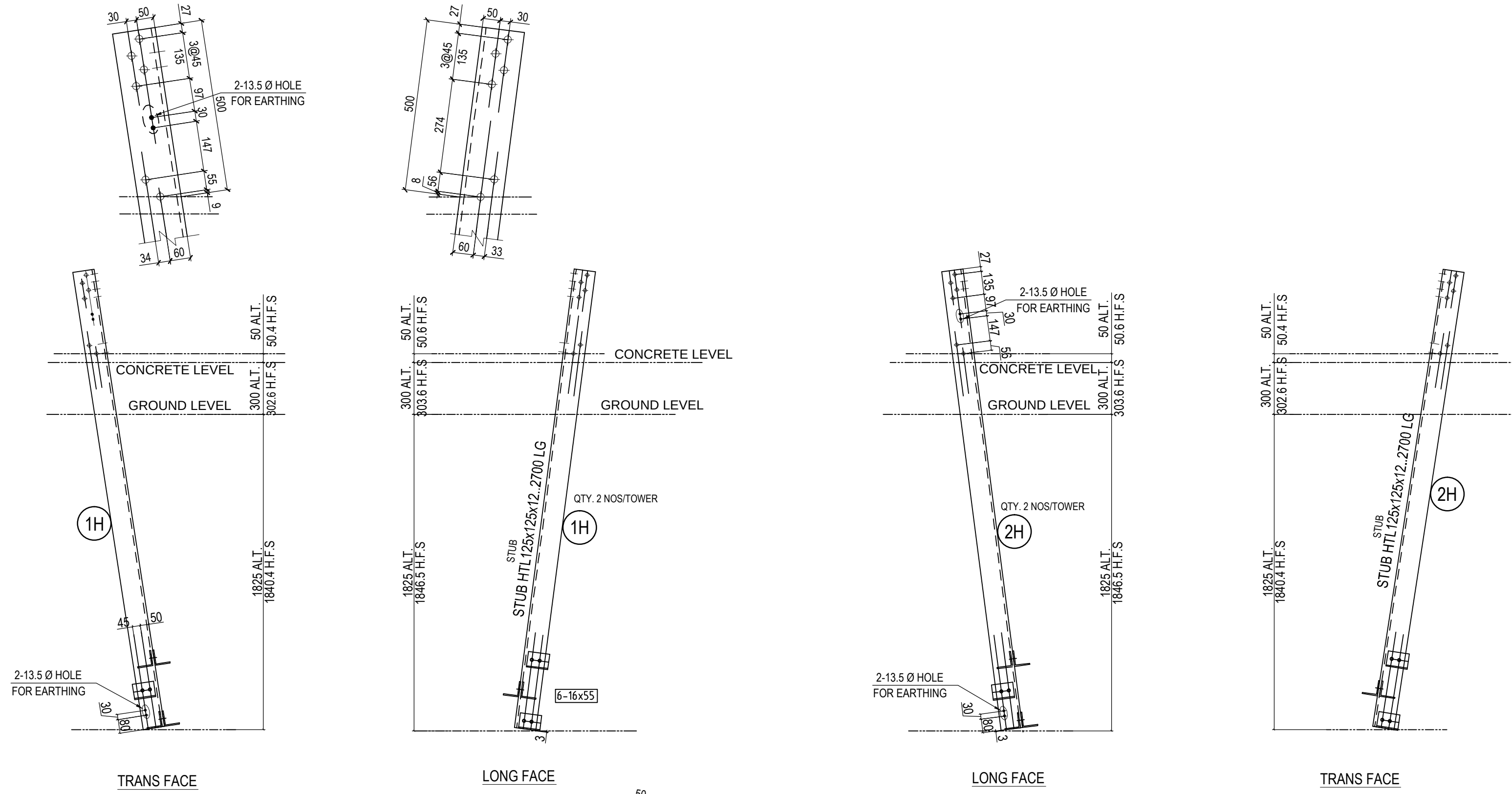
	Name:	Date:
Prepared:	P.R.C. / N.R.A.	25.01.2017
Drawn:	P.R.C.	25.01.2017
Checked:	P.S.CHOWDHRY	25.01.2017
Approved:	M.B.NENE	25.01.2017

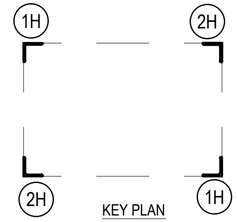
Drawing Detail:
Project: Karuma Interconnection, 400 KV Karuma - Kawanda Transmission Line

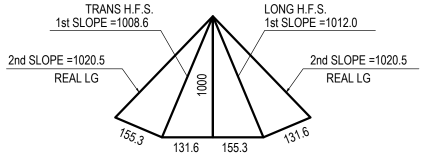
Title: FOUNDATION SKETCH FOR TT "DA" (0°-2°)-GOOD SOIL

Revision:		
D		
C		
B	Revised as per vide mail dated 27.02.2017	18.03.2017
A	Revised as per vide letter KPTL_SH_2016_3134 dated 07.01.2017	25.01.2017
No.	0	21.12.2016

Scale: | Contractor's Drawing No.: - - - - - - - - - - 2 9 5 / F D N - 1 | Sheet No. 12 | Total Sh. 17

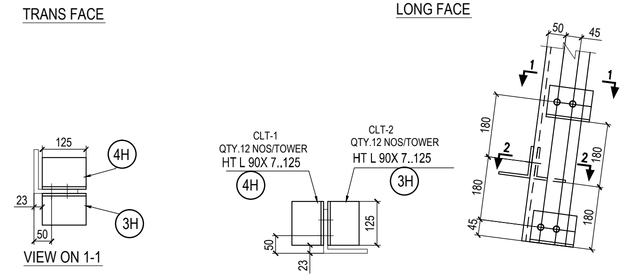






SLOPE DIAGRAM
FOR BOTTOM PART

M16 H.R.H. Bolts, Nuts and Washers with 3% extra	
SIZE	QTY./TOWER
55 mm long bolt	25
Spring Washers	25
Std. Flat Washers	25



VIEW ON 1-1

VIEW ON 2-2

NOTES:-

1). All dimensions are in mm.
2). All holes are 17.5 Ø for 16 Ø bolts noted as ○
3). Mild steel shall conform to BSEN 10025 GR-S 235 JO.
4). High tensile steel shall conform to BSEN 10025 GR-S 355 JO.
5). All steel members are to be hot dip galvanized according to ISO 1461 with uniform zinc coating of min.610 g/m ^{2} & 86μm. thk.
6). Standard spring & flat washers as per DIN 127 (Type B) & DIN 126 respectively to be supplied with each bolt.
7). Min. rolled & cut edge security shall be 22 & 26mm respectively for 16Ø bolts.
8). All item nos are to be prefixed with letter 295DA.
9). All bolts and nuts conforming to ISO 898 (PART 1 & 2) with mechanical properties class 6.8 / 6.
10). All bolts, nuts and washers are to be hot dip galvanized according to ASTM A153.
11). B / N comprises of 1 bolt + 1 nut + 1 spring washer & 1 flat washer.

REFERENCE DRAWINGS	CLIENTS DOCUMENT NO.	REF DRG No.
+0.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5500-K-K-DD-STW-054	295/DA-8
+0.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5500-K-K-DD-STW-055	295/DA-8
+1.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5500-K-K-DD-STW-056	295/DA-9
+1.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5500-K-K-DD-STW-057	295/DA-9
+2.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-058	295/DA-10
+2.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-059	295/DA-10
+3.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-060	295/DA-11
+3.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-061	295/DA-11
+4.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-062	295/DA-12
+4.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-063	295/DA-12
STUB SETTING TEMPLATE	KIP-L-5900-K-K-DD-STW-064	295/DA-2

APPROVAL REFERENCE LETTER No.

L.D.	M.D.	ITEM NO :-
SANDIP 01.12.2016		1 TO 3

Reference Drawings

Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: ____________ Signature / Stamp ____________ Project Consultant

"Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

"CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP"

Project Drawing: KIP-L-5500-K-K-DD-STW-068-B

	Name:	Date:
Prepared:	HITESH	21.11.2016
Drawn:	SACHIN	21.11.2016
Checked:	N.R.A.	21.11.2016
Approved:	M.B.NENE	21.11.2016

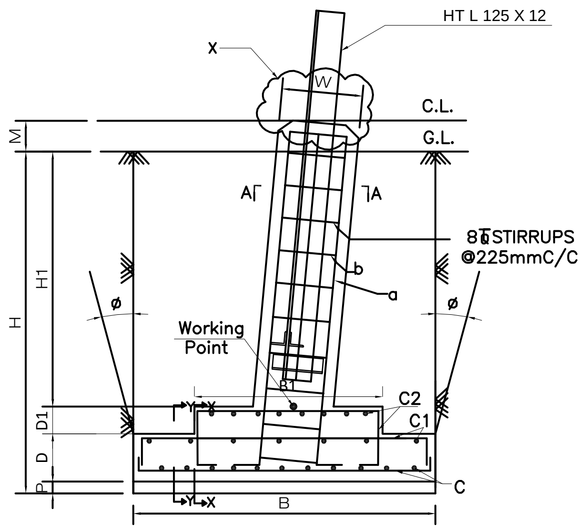
Drawing Detail:
Project: "Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line"

Revision:		
D		
C		
B	STUB LENGTH REVISED	25.01.2017
A	Ref. No. KIP/L/5500/INK/B-0365	04.01.2017
No.	0	01.12.2016

Title: TOWER TYPE "DA" (0°-2°)
STRUCTURAL DRAWING OF
STUB

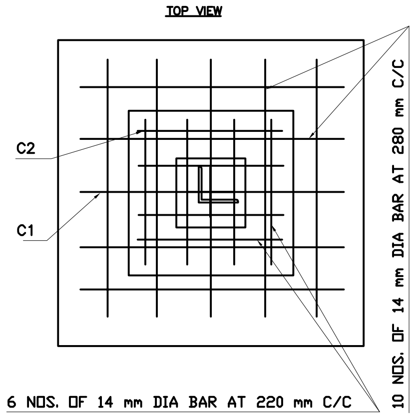
Scale: 1 : 25	"Contractor's Drawing No."	--------295/DA-01	Sheet No. 1	Total Sh. 1

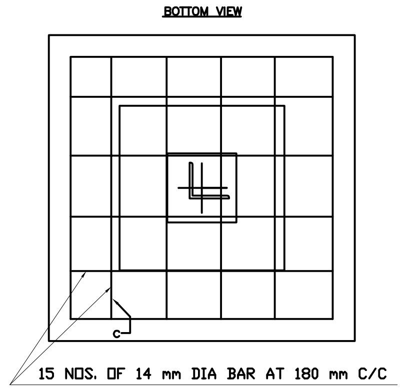
PAPER SIZE A2



FDN DIM TABLE

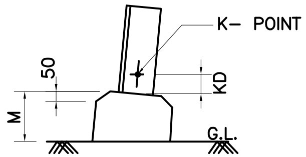
H	3500
B	2640
W	550
M	300
P	50
D	350
D1	250
H1	2850
B1	1200
KD	50



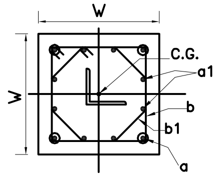


BAR BENDING DETAIL

MARK	SHAPE & SIZE	DIA	UNIT WT KG/M	LENGTH	QTY / LEG	STEEL FY 500
C2	422 1100 422, 150 150	14 T	1.21	2244	2 X 6	32.58
C1	170 2540 170	14 T	1.21	2880	2 X 10	69.70
C	150 2540 150	14 T	1.21	2840	2 X 15	103.09
a	300 3625	20 T	2.469	3925	4	38.76
a1	300 3625	16 T	1.578	3925	8	49.55
b	450 450	8 T	0.395	1950	17	13.09
b1	160 205	8 T	0.395	1610	17	10.81
				TOTAL WEIGHT PER LEG =		317.58kg



DETAIL AT X



SECTION A-A

NOTES:-

1 ALL DIMENSIONS ARE IN mm, UNLESS OTHERWISE SPECIFIED.

2 STEEL USED FOR REINFORCEMENT IS OF GRADE 500. STEEL USED FOR STIRRUPS IS OF GRADE 500.

3 GRADE OF CONCRETE USED IS C25 AS PER SPECIFICATION.

4 MINIMUM COVER TO MAIN REINFORCEMENT SHALL BE 100 mm TO BOTTOM SURFACE & 50 mm TO TOP OF SIDE SURFACE AS PER SPECIFICATION.

5 NO FOUNDATION SHALL REST ON FILLED UP SOIL.

6 NOT MORE THAN 50% BARS SHALL BE LAPPED AT ONE SECTION UNLESS SPECIFIED / SHOWN.

7 ALL HOOKS, BENDS, LAPS, SPLICES & DEVELOPMENT LENGTH SHALL BE AS PER BS: 8110-1985 EXCEPT STATED OTHERWISE.

8 DRAWING NOT TO SCALE

QUANTITY/TOWER

CONC.VOL M^3	C 25	15.009
	C 10	1.394
EXCAVATION.VOL m^3		97.575
STEEL KG	FY 500	1271

Reference Drawings

Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: ____________ Signature / Stamp ____________

Project Consultant

Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

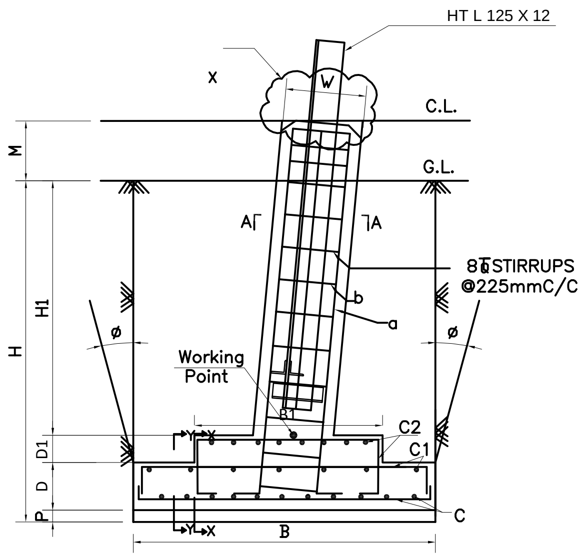
CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: K I P - L - 5 8 0 0 - K - K - D D - C F D - 0 0 1 - B

	Name:	Date:	Drawing Detail:
Prepared:	P.R.C. / N.R.A.	25.01.2017	Project:
Drawn:	P.R.C.	25.01.2017	Karuma Interconnection
Checked:	P.S.CHOWDHRY	25.01.2017	400 KV Karuma - Kawanda Transmission Line
Approved:	M.B.NENE	25.01.2017	
			Title:
Revision: D			
C			
B	Revised as per vide mail dated 27.02.2017	18.03.2017	FOUNDATION SKETCH FOR TT "DA" (0°-2°)-GOOD SOIL
A	Revised as per vide letter KPTL_SH_2016_3134 dated 07.01.2017	25.01.2017	
No.	0	21.12.2016	

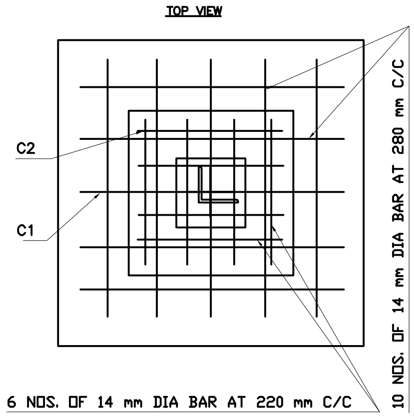
Scale:	Contractor's Drawing No.	- - - - - - - - 2 9 5 / F D N - 1	Sheet No. 12	Total Sh. 17

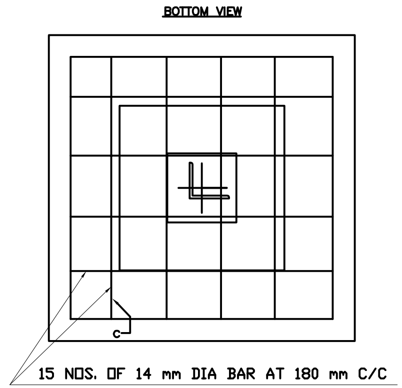
(0.5M RAISED CHIMNEY)



FDN DIM TABLE

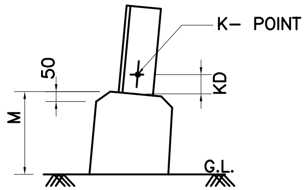
H	3500
B	2640
W	550
M	800
P	50
D	350
D1	250
H1	2850
B1	1200
KD	50



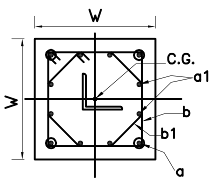


BAR BENDING DETAIL

MARK	SHAPE & SIZE	DIA	UNIT WT KG/M	LENGTH	QTY / LEG	STEEL FY 500
C2	1100, 422, 150, 150, 422	14	1.21	2244	2 X 6	32.58
C1	2540, 170, 170	14	1.21	2880	2 X 10	69.70
C	150, 2540, 150	14	1.21	2840	2 X 15	103.09
a	300, 4135	20	2.469	4435	4	43.80
a1	300, 4135	16	1.578	4435	8	55.99
b	450, 450	8	0.395	1950	19	14.63
b1	160, 205	8	0.395	1610	19	12.08
				TOTAL WEIGHT PER LEG =		331.87kg



DETAIL AT X



SECTION A-A

QUANTITY/TOWER

CONC.VOL M^3	C 25	15.614
	C 10	1.394
EXCAVATION.VOL m^3		97.575
STEEL KG	FY 500	1328

NOTES:-

1 ALL DIMENSIONS ARE IN mm, UNLESS OTHERWISE SPECIFIED.

2 STEEL USED FOR REINFORCEMENT IS OF GRADE 500. STEEL USED FOR STIRRUPS IS OF GRADE 500.

3 GRADE OF CONCRETE USED IS C25 AS PER SPECIFICATION.

4 MINIMUM COVER TO MAIN REINFORCEMENT SHALL BE 100 mm TO BOTTOM SURFACE & 50 mm TO TOP OF SIDE SURFACE AS PER SPECIFICATION.

5 NO FOUNDATION SHALL REST ON FILLED UP SOIL.

6 NOT MORE THAN 50% BARS SHALL BE LAPPED AT ONE SECTION UNLESS SPECIFIED / SHOWN.

7 ALL HOOKS, BENDS, LAPS, SPLICES & DEVELOPMENT LENGTH SHALL BE AS PER BS: 8110-1985 EXCEPT STATED OTHERWISE.

8 DRAWING NOT TO SCALE

Reference Drawings

Drawing No:	Title:

Approved / Approved with Comments / Not Approved / For Information

Date: Signature / Stamp

Project Consultant

Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA 中国水电 SINOHYDRO

CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: K I P - L - 5 8 0 0 - K - K - D D - C F D - 0 0 1 - B

Drawing Detail:

	Name:	Date:
Prepared:	P.R.C. / N.R.A.	25.01.2017
Drawn:	P.R.C.	25.01.2017
Checked:	P.S.CHOWDHRY	25.01.2017
Approved:	M.B.NENE	25.01.2017

Project:
Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line

Title:
FOUNDATION SKETCH FOR TT "DA" (0°-2°)-GOOD SOIL
(0.5M RAISED CHIMNEY)

Revision:		
D		
C		
B	Revised as per vide mail dated 27.02.2017	18.03.2017
A	Revised as per vide letter KPTL_SH_2016_3134 dated 07.01.2017	25.01.2017
No.	0	21.12.2016

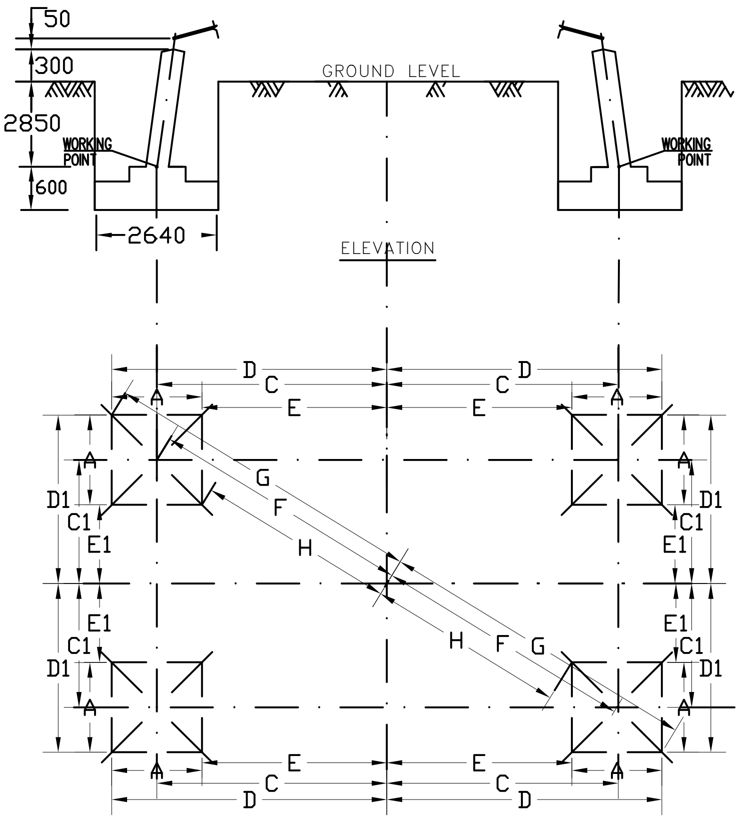
Scale:

Contractor's Drawing No. - - - - - - - - - 2 9 5 / F D N - 1

Sheet No. 13 | Total Sh. 17

SHEET NO.: 14 OF 17
DESIGN NO: 295/FDN-1
TOWER TYPE : TT DA
SOIL TYPE : GOOD SOIL

TYPICAL EXCAVATION DETAIL



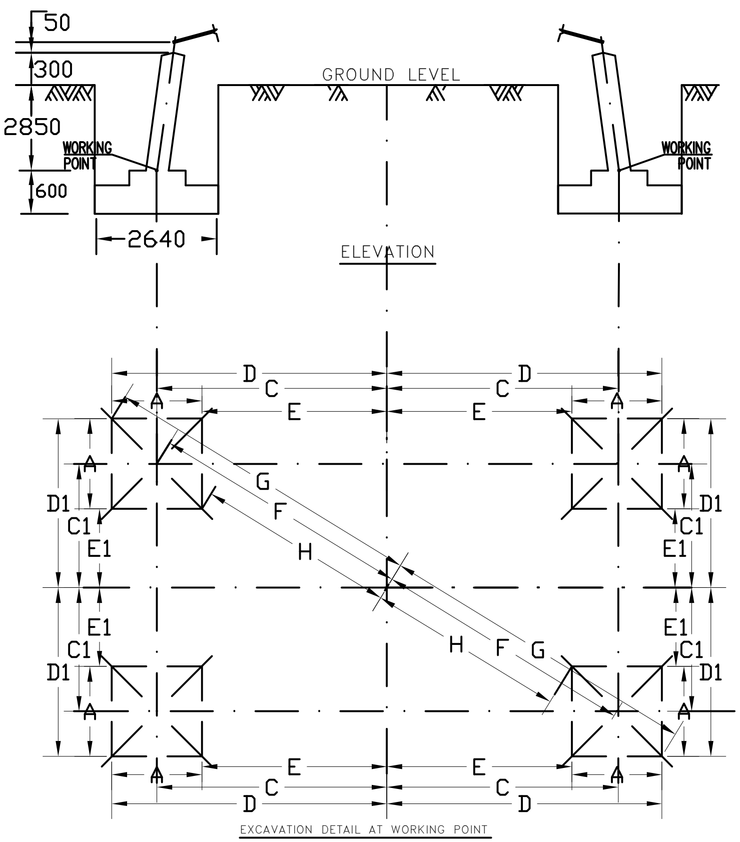
EXCAVATION DETAIL AT WORKING POINT

EXCAVATION DIMENSION TABLE

SOIL TYPE	TOWER TYPE	A	C	C1	D	D1	E	E1	F	G	H
GOOD SOIL	BTB + 0M LE	2640	5970	5320	7290	6640	4650	4000	7997	9861	6134
GOOD SOIL	BTB + 1M LE	2640	6125	5452	7445	6772	4805	4132	8200	10065	6338
GOOD SOIL	BTB + 2M LE	2640	6280	5583	7600	6903	4960	4263	8403	10268	6541
GOOD SOIL	BTB + 3M LE	2640	6435	5715	7755	7035	5115	4395	8607	10471	6744
GOOD SOIL	BTB + 4M LE	2640	6591	5847	7911	7167	5271	4527	8811	10675	6949
GOOD SOIL	BTB + 3M BE + 0M LE	2640	6435	5715	7755	7035	5115	4395	8607	10471	6744
GOOD SOIL	BTB + 3M BE + 1M LE	2640	6591	5847	7911	7167	5271	4527	8811	10675	6949
GOOD SOIL	BTB + 3M BE + 2M LE	2640	6746	5978	8066	7298	5426	4658	9014	10878	7152
GOOD SOIL	BTB + 3M BE + 3M LE	2640	6901	6110	8221	7430	5581	4790	9218	11082	7355
GOOD SOIL	BTB + 3M BE + 4M LE	2640	7056	6241	8376	7561	5736	4921	9421	11284	7558
GOOD SOIL	BTB + 6M BE + 0M LE	2640	6901	6110	8221	7430	5581	4790	9218	11082	7355
GOOD SOIL	BTB + 6M BE + 1M LE	2640	7056	6241	8376	7561	5736	4921	9421	11284	7558
GOOD SOIL	BTB + 6M BE + 2M LE	2640	7212	6373	8532	7693	5892	5053	9625	11489	7762
GOOD SOIL	BTB + 6M BE + 3M LE	2640	7367	6504	8687	7824	6047	5184	9828	11691	7965
GOOD SOIL	BTB + 6M BE + 4M LE	2640	7522	6636	8842	7956	6202	5316	10031	11895	8169

SHEET NO.: 15 OF 17
DESIGN NO: 295/FDN-1
TOWER TYPE : TT DA
SOIL TYPE : GOOD SOIL

TYPICAL EXCAVATION DETAIL



EXCAVATION DIMENSION TABLE

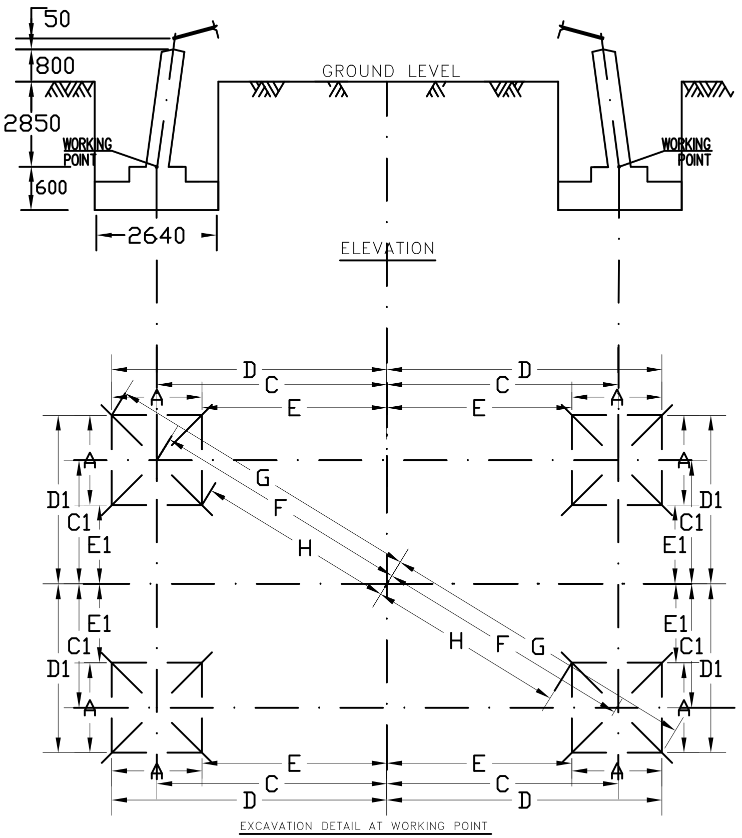
SOIL TYPE	TOWER TYPE	A	C	C1	D	D1	E	E1	F	G	H
GOOD SOIL	BTB + 9M BE + 0M LE	2640	7367	6504	8687	7824	6047	5184	9828	11691	7965
GOOD SOIL	BTB + 9M BE + 1M LE	2640	7522	6636	8842	7956	6202	5316	10031	11895	8169
GOOD SOIL	BTB + 9M BE + 2M LE	2640	7677	6768	8997	8088	6357	5448	10235	12099	8373
GOOD SOIL	BTB + 9M BE + 3M LE	2640	7833	6899	9153	8219	6513	5579	10439	12302	8576
GOOD SOIL	BTB + 9M BE + 4M LE	2640	7988	7031	9308	8351	6668	5711	10642	12506	8780
GOOD SOIL	BTB + 12M BE + 0M LE	2640	7833	6899	9153	8219	6513	5579	10439	12302	8576
GOOD SOIL	BTB + 12M BE + 1M LE	2640	7988	7031	9308	8351	6668	5711	10642	12506	8780
GOOD SOIL	BTB + 12M BE + 2M LE	2640	8143	7162	9463	8482	6823	5842	10845	12708	8983
GOOD SOIL	BTB + 12M BE + 3M LE	2640	8298	7294	9618	8614	6978	5974	11049	12912	9186
GOOD SOIL	BTB + 12M BE + 4M LE	2640	8454	7426	9774	8746	7134	6106	11253	13116	9391

B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) FOR TRANSVERSE FACE = 11015 mm
B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) FOR LONGITUDINAL FACE = 9868 mm

(0.5M RAISED CHIMNEY)

SHEET NO.: 16 OF 17
DESIGN NO: 295/FDN-1
TOWER TYPE : TT DA
SOIL TYPE : GOOD SOIL
(0.5m Raised Chimney)

TYPICAL EXCAVATION DETAIL



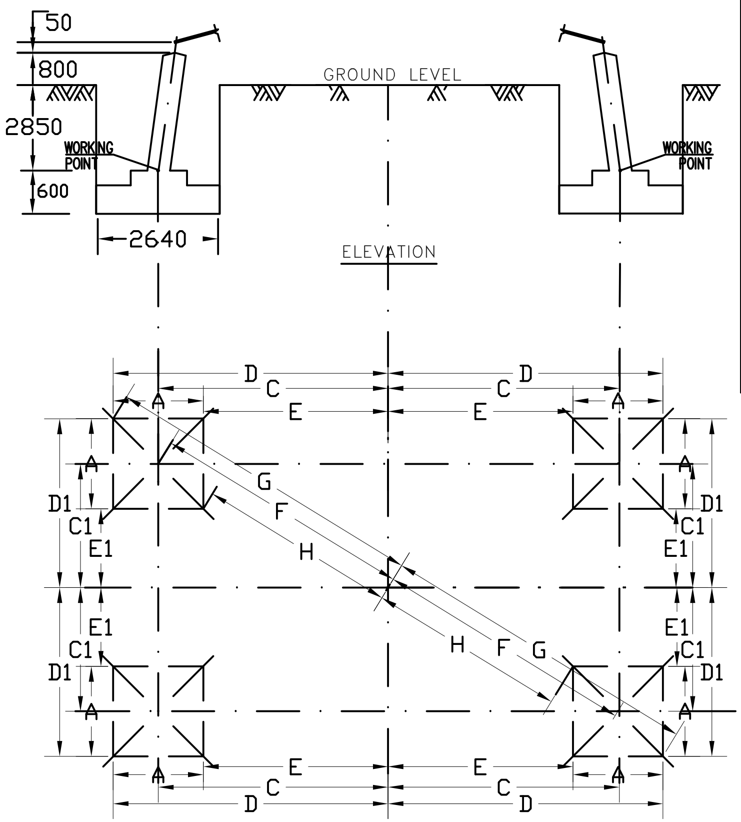
EXCAVATION DIMENSION TABLE

SOIL TYPE	TOWER TYPE	A	C	C1	D	D1	E	E1	F	G	H
GOOD SOIL	BTB + 0M LE	2640	6047	5386	7367	6706	4727	4066	8098	9963	6236
GOOD SOIL	BTB + 1M LE	2640	6202	5518	7522	6838	4882	4198	8302	10166	6439
GOOD SOIL	BTB + 2M LE	2640	6358	5649	7678	6969	5038	4329	8506	10370	6643
GOOD SOIL	BTB + 3M LE	2640	6513	5781	7833	7101	5193	4461	8709	10573	6847
GOOD SOIL	BTB + 4M LE	2640	6668	5912	7988	7232	5348	4592	8912	10776	7049
GOOD SOIL	BTB + 3M BE + 0M LE	2640	6513	5781	7833	7101	5193	4461	8709	10573	6847
GOOD SOIL	BTB + 3M BE + 1M LE	2640	6668	5912	7988	7232	5348	4592	8912	10776	7049
GOOD SOIL	BTB + 3M BE + 2M LE	2640	6823	6044	8143	7364	5503	4724	9116	10979	7253
GOOD SOIL	BTB + 3M BE + 3M LE	2640	6979	6176	8299	7496	5659	4856	9320	11184	7457
GOOD SOIL	BTB + 3M BE + 4M LE	2640	7134	6307	8454	7627	5814	4987	9523	11387	7660
GOOD SOIL	BTB + 6M BE + 0M LE	2640	6979	6176	8299	7496	5659	4856	9320	11184	7457
GOOD SOIL	BTB + 6M BE + 1M LE	2640	7134	6307	8454	7627	5814	4987	9523	11387	7660
GOOD SOIL	BTB + 6M BE + 2M LE	2640	7289	6439	8609	7759	5969	5119	9726	11590	7864
GOOD SOIL	BTB + 6M BE + 3M LE	2640	7444	6570	8764	7890	6124	5250	9929	11793	8067
GOOD SOIL	BTB + 6M BE + 4M LE	2640	7600	6702	8920	8022	6280	5382	10133	11997	8271

(0.5M RAISED CHIMNEY)

SHEET NO.: 17 OF 17
DESIGN NO: 295/FDN-1
TOWER TYPE : TT DA
SOIL TYPE : GOOD SOIL
(0.5m Raised Chimney)

TYPICAL EXCAVATION DETAIL

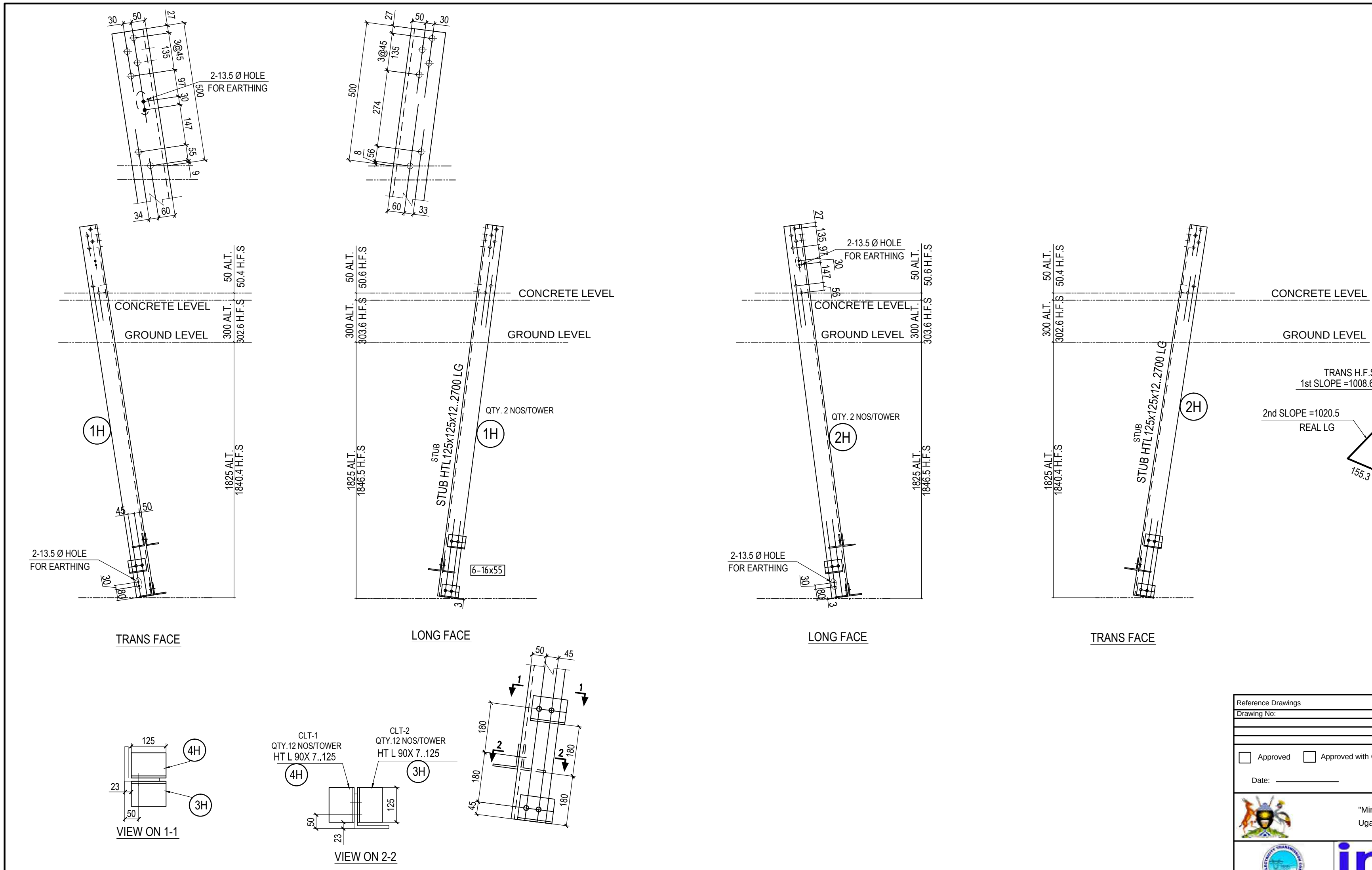


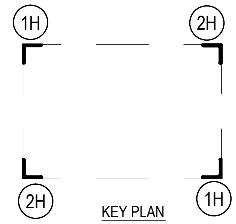
EXCAVATION DETAIL AT WORKING POINT

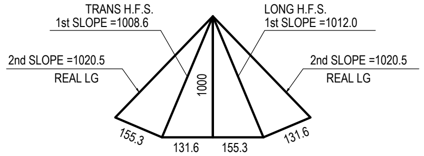
EXCAVATION DIMENSION TABLE

SOIL TYPE	TOWER TYPE	A	C	C1	D	D1	E	E1	F	G	H
GOOD SOIL	BTB + 9M BE + 0M LE	2640	7444	6570	8764	7890	6124	5250	9929	11793	8067
GOOD SOIL	BTB + 9M BE + 1M LE	2640	7600	6702	8920	8022	6280	5382	10133	11997	8271
GOOD SOIL	BTB + 9M BE + 2M LE	2640	7755	6833	9075	8153	6435	5513	10336	12200	8474
GOOD SOIL	BTB + 9M BE + 3M LE	2640	7910	6965	9230	8285	6590	5645	10540	12403	8678
GOOD SOIL	BTB + 9M BE + 4M LE	2640	8065	7097	9385	8417	6745	5777	10743	12607	8881
GOOD SOIL	BTB + 12M BE + 0M LE	2640	7910	6965	9230	8285	6590	5645	10540	12403	8678
GOOD SOIL	BTB + 12M BE + 1M LE	2640	8065	7097	9385	8417	6745	5777	10743	12607	8881
GOOD SOIL	BTB + 12M BE + 2M LE	2640	8221	7228	9541	8548	6901	5908	10947	12811	9085
GOOD SOIL	BTB + 12M BE + 3M LE	2640	8376	7360	9696	8680	7056	6040	11151	13014	9289
GOOD SOIL	BTB + 12M BE + 4M LE	2640	8531	7491	9851	8811	7211	6171	11354	13217	9492

B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) FOR TRANSVERSE FACE = 11015 mm
B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) FOR LONGITUDINAL FACE = 9868 mm







SLOPE DIAGRAM
FOR BOTTOM PART

M16 H.R.H. Bolts, Nuts and Washers with 3% extra	
SIZE	QTY./TOWER
55 mm long bolt	25
Spring Washers	25
Std. Flat Washers	25

NOTES:-

1). All dimensions are in mm.
2). All holes are 17.5 Ø for 16 Ø bolts noted as ○
3). Mild steel shall conform to BSEN 10025 GR-S 235 JO.
4). High tensile steel shall conform to BSEN 10025 GR-S 355 JO.
5). All steel members are to be hot dip galvanized according to ISO 1461 with uniform zinc coating of min.610 g/m^2 & 86µm. thk.
6). Standard spring & flat washers as per DIN 127 (Type B) & DIN 126 respectively to be supplied with each bolt.
7). Min. rolled & cut edge security shall be 22 & 26mm respectively for 16Ø bolts.
8). All item nos are to be prefixed with letter 295DA.
9). All bolts and nuts conforming to ISO 898 (PART 1 & 2) with mechanical properties class 6.8 / 6.
10). All bolts, nuts and washers are to be hot dip galvanized according to ASTM A153.
11). B / N comprises of 1 bolt + 1 nut + 1 spring washer & 1 flat washer.

REFERENCE DRAWINGS	CLIENTS DOCUMENT NO.	REF DRG No.
+0.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5500-K-K-DD-STW-054	295/DA-8
+0.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5500-K-K-DD-STW-055	295/DA-8
+1.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5500-K-K-DD-STW-056	295/DA-9
+1.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5500-K-K-DD-STW-057	295/DA-9
+2.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-058	295/DA-10
+2.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-059	295/DA-10
+3.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-060	295/DA-11
+3.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-061	295/DA-11
+4.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-062	295/DA-12
+4.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-063	295/DA-12
STUB SETTING TEMPLATE	KIP-L-5900-K-K-DD-STW-064	295/DA-2

APPROVAL REFERENCE LETTER No.

L.D.	M.D.	ITEM NO :-
SANDIP 01.12.2016		1 TO 3

Reference Drawings

Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: ____________ Signature / Stamp ____________ Project Consultant

"Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

"CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP"

Project Drawing: KIP-L-5500-K-K-DD-STW-068-B

	Name:	Date:
Prepared:	HITESH	21.11.2016
Drawn:	SACHIN	21.11.2016
Checked:	N.R.A.	21.11.2016
Approved:	M.B.NENE	21.11.2016

Drawing Detail:
Project: "Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line"

Revision:		
D		
C		
B	STUB LENGTH REVISED	25.01.2017
A	Ref. No. KIP/L/5500/INK/B-0365	04.01.2017
No.	0	01.12.2016

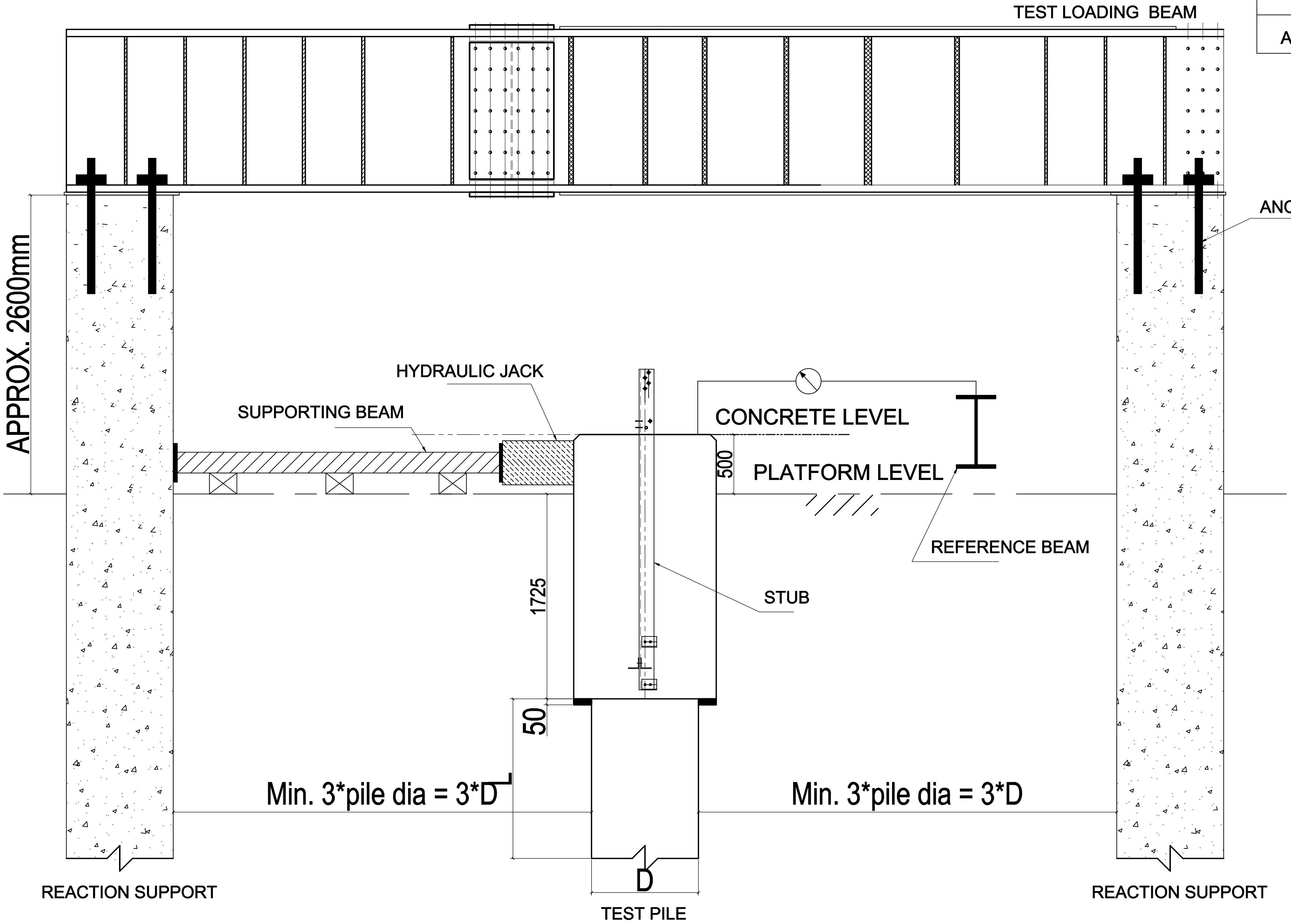
Title: TOWER TYPE "DA" (0°-2°)
STRUCTURAL DRAWING OF
STUB

Scale: 1 : 25 — "Contractor's Drawing No." 295/DA-01 — Sheet No. 1 — Total Sh. 1

PAPER SIZE A2

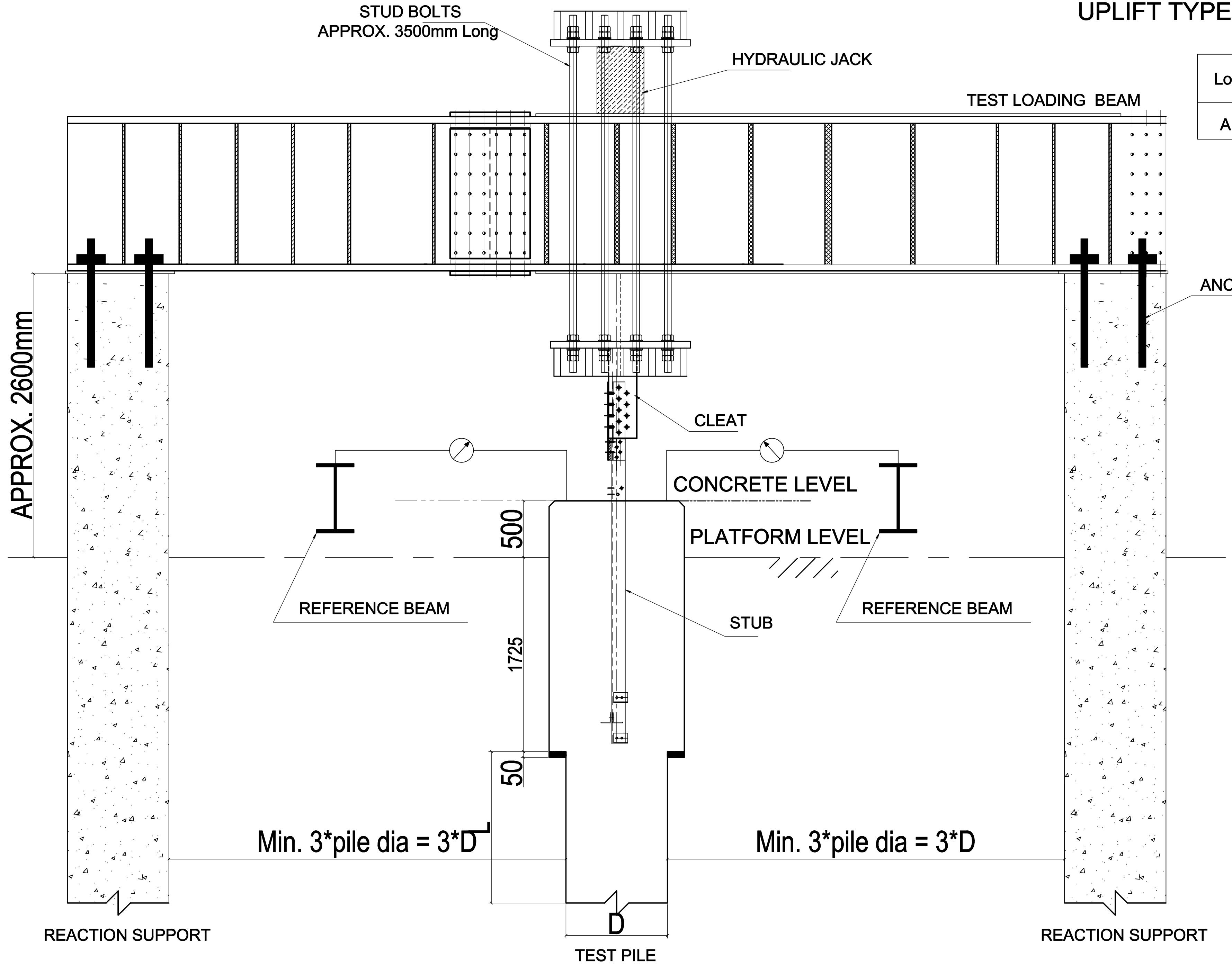
ANNEXURE - 1 (A)
LATERAL TYPE TEST ARRANGEMENT

Location No.	Diameter 'D'	Length 'L'
AP104/5	900mm	12775mm



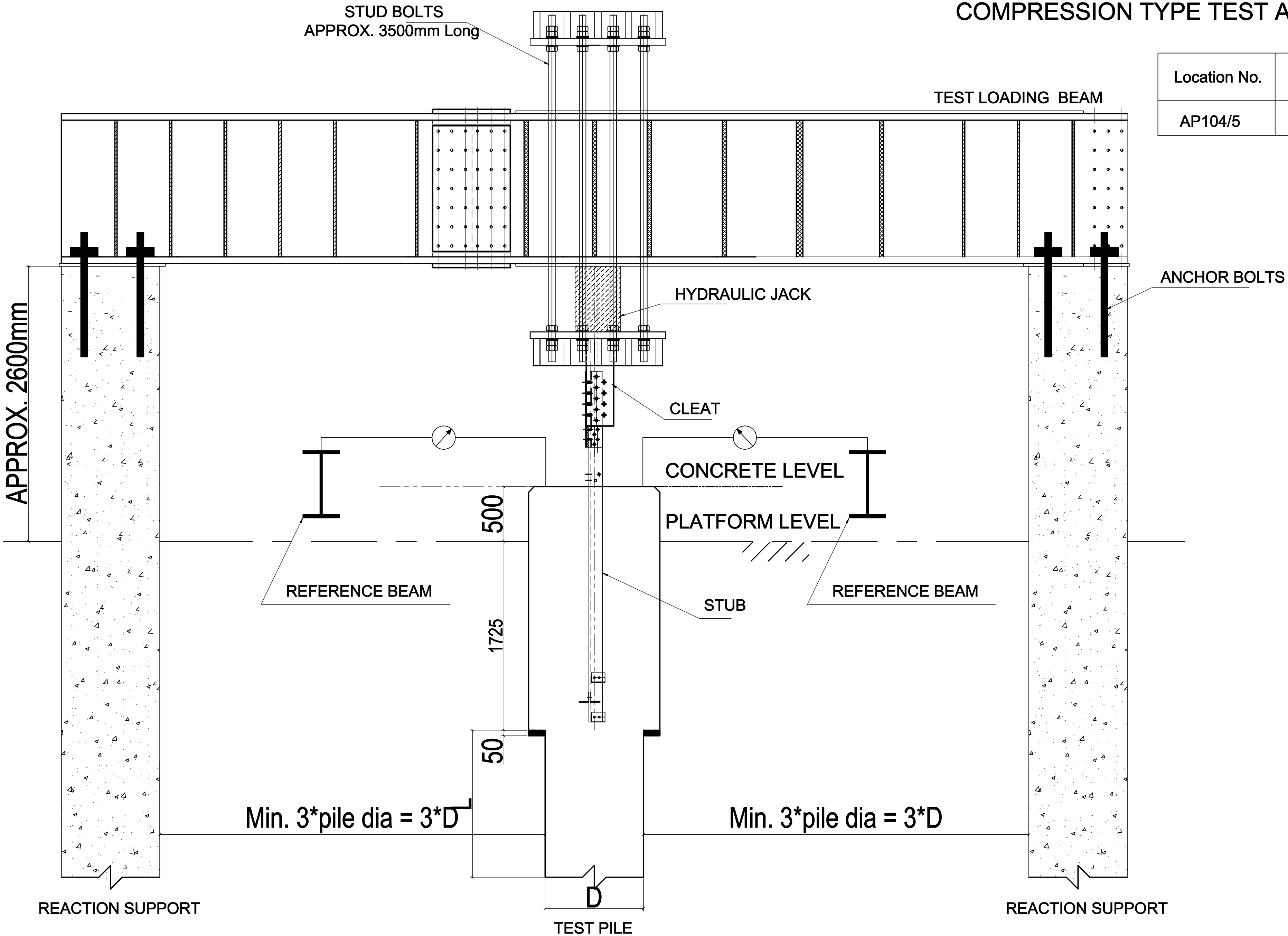
ANNEXURE - 1 (B)
UPLIFT TYPE TEST ARRANGEMENT

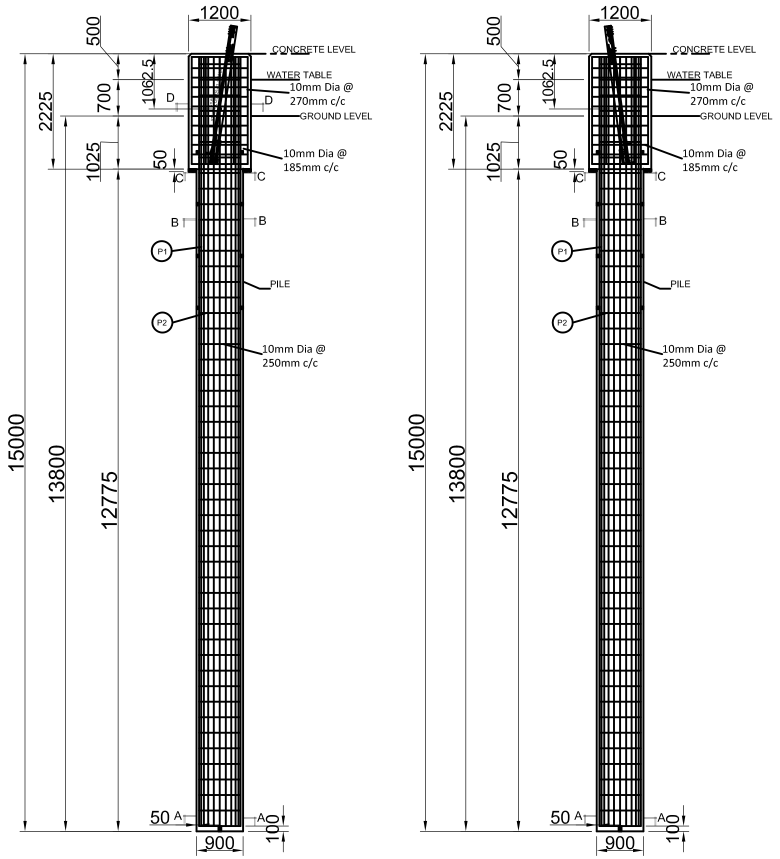
Location No.	Diameter 'D'	Length 'L'
AP104/5	900mm	12775mm



ANNEXURE - 1 (C)
COMPRESSION TYPE TEST ARRANGEMENT

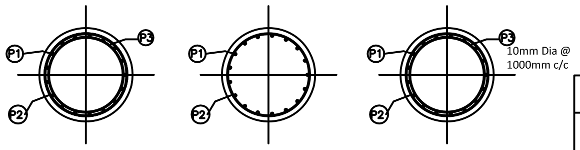
Location No.	Diameter 'D'	Length 'L'
AP104/5	900mm	12775mm





TRANSVERSE FACE

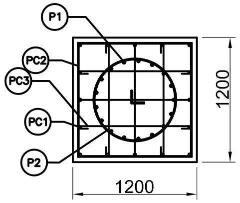
LONGITUDINAL FACE



SECTION A-A
SCALE - 1:50

SECTION B-B
SCALE - 1:50

SECTION C-C
SCALE - 1:50

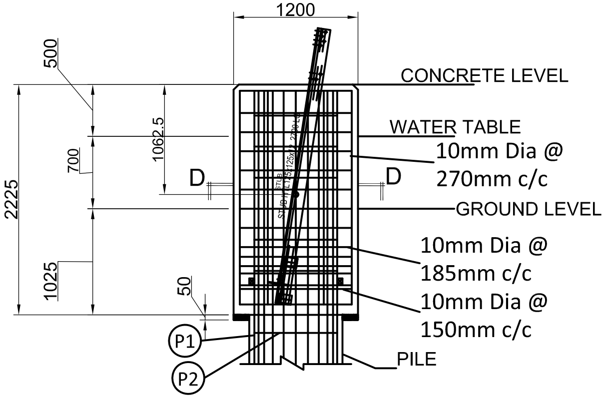


SECTION D-D
SCALE - 1:50

BAR BENDING SCHEDULE (PER PILE)

LOCATION	MARK	SHAPE AND SIZE	LENGTH (MM)	DIA. (MM)	WEIGHT (KG/M)	QTY. PER LEG	TOTAL WEIGHT (KG)
PILE	P1	14850	14850	25	3.858	17	973.96
	P2		2515	10	0.617	60	93.11
	P3		2295	10	0.617	13	18.40
						TOTAL (A)	1085.47
PILE CAP	PC1	2075	2075	10	0.617	16	20.49
	PC2	1100 75 75 1100	4550	10	0.617	12	33.69
	PC3	1100 75 75	1250	10	0.617	12	9.26
						TOTAL (B)	63.44
						TOTAL (A+B) (KG)	1148.91

QUANTITES PER TOWER		
PILE	CONCRETE C30 GRADE(CU.M.)	32.51
	BORING (R.M.T)	51.1
	REBARS Fe460 GRADE (KG)	4341.88
PILE CAP	CONCRETE C30 GRADE(CU.M.)	12.82
	LEAN CONC. C10 GRADE (CU.M)	0.16
	EXCAVATION (CU.M)	5.91
	REBARS Fe460 GRADE (KG)	253.76



ENLARGE VIEW OF PILE CAP
SCALE - 1:50

NOTES:-

1 ALL DIMENSIONS ARE IN mm, UNLESS OTHERWISE SPECIFIED.

2 STEEL USED FOR REINFORCEMENT IS OF GRADE 460. STEEL USED FOR STIRRUPS IS OF GRADE 460.

3 GRADE OF CONCRETE USED IS C30 AS PER SPECIFICATION.

4 MINIMUM COVER TO MAIN REINFORCEMENT SHALL BE 100 mm TO BOTTOM SURFACE & 50 mm TO TOP & SIDE SURFACE AS PER SPECIFICATION.

5 NO FOUNDATION SHALL REST ON FILLED UP SOIL.

6 NOT MORE THAN 50% BARS SHALL BE LAPPED AT ONE SECTION UNLESS SPECIFIED / SHOWN.

7 ALL HOOKS, BENDS, LAPS, SPLICES & DEVELOPMENT LENGTH SHALL BE AS PER BS: 8110-1985 EXCEPT STATED OTHERWISE.

8 DRAWING NOT TO SCALE

Reference Drawings

Drawing No:	Title:

Approved / Approved with Comments / Not Approved / For Information

Date: Signature / Stamp

Project Consultant

Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

intec
GOPA International Energy Consultants

中国电建 POWERCHINA
中国水电 SINOHYDRO

CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: K I P - L - 5 8 0 0 - K - K - D D - C F D - 0 6 6 - A

	Name:	Date:
Prepared:	V.J.A / N.R.A.	27.12.2018
Drawn:	V.J.A	27.12.2018
Checked:	P.S.CHOWDHRY	27.12.2018
Approved:	M.B.NENE	27.12.2018

Drawing Detail:

Project:
Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line

Title:
PILE FOUNDATION SKETCH (SINGLE PILE)
FOR TT "DA" (0°-2°)
(BASIC BODY + 3mBE + 0mLE)FOR LOCATION AP 104/5

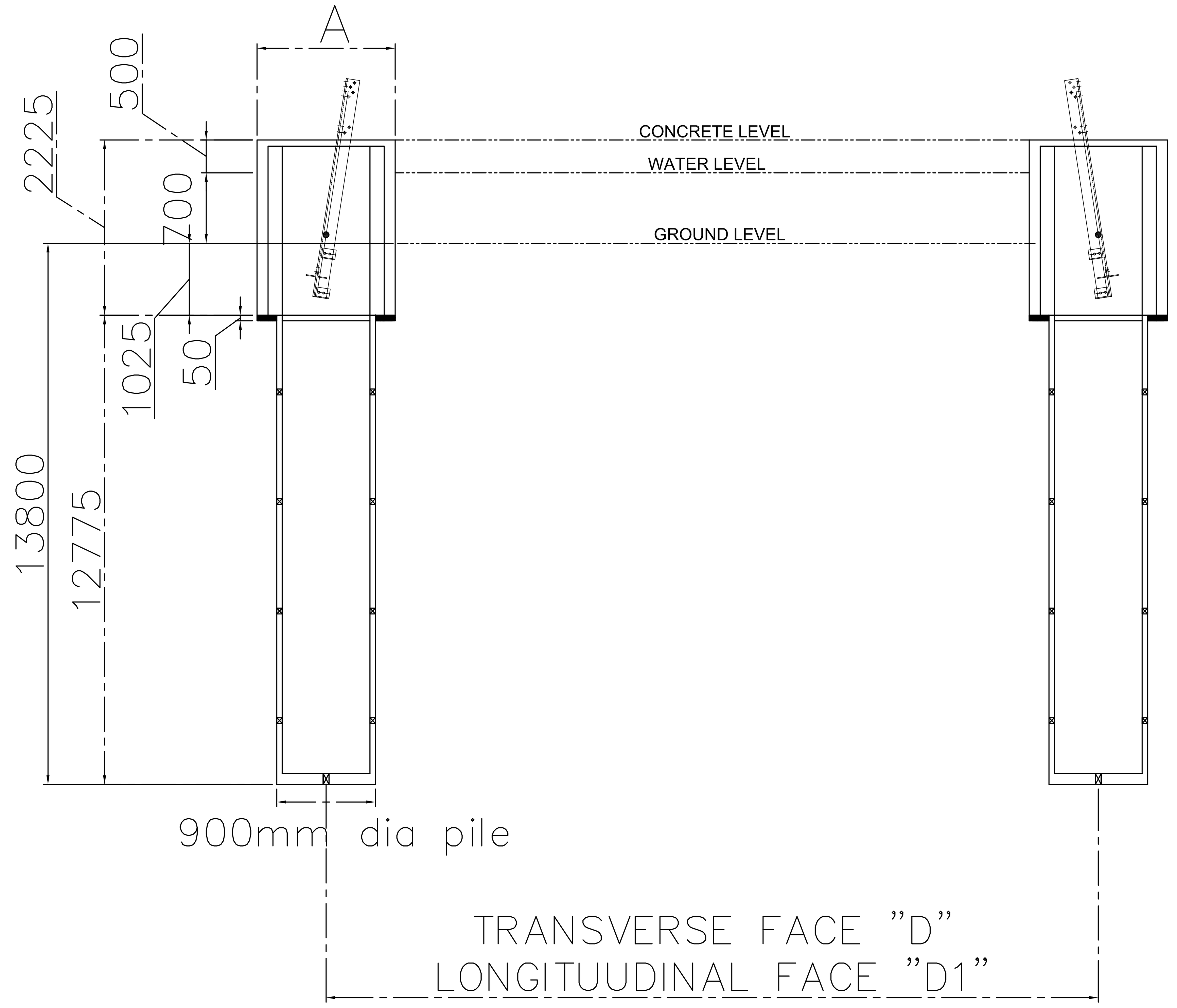
Revision:		
D		
C		
B		
A	Ref. No. KIP/L/5800/INK/ 8-3335	14.02.2019
No.	0	27.12.2018

Scale:

Contractor's Drawing No.: - - - - - - - 2 9 5 / F D N - P 2

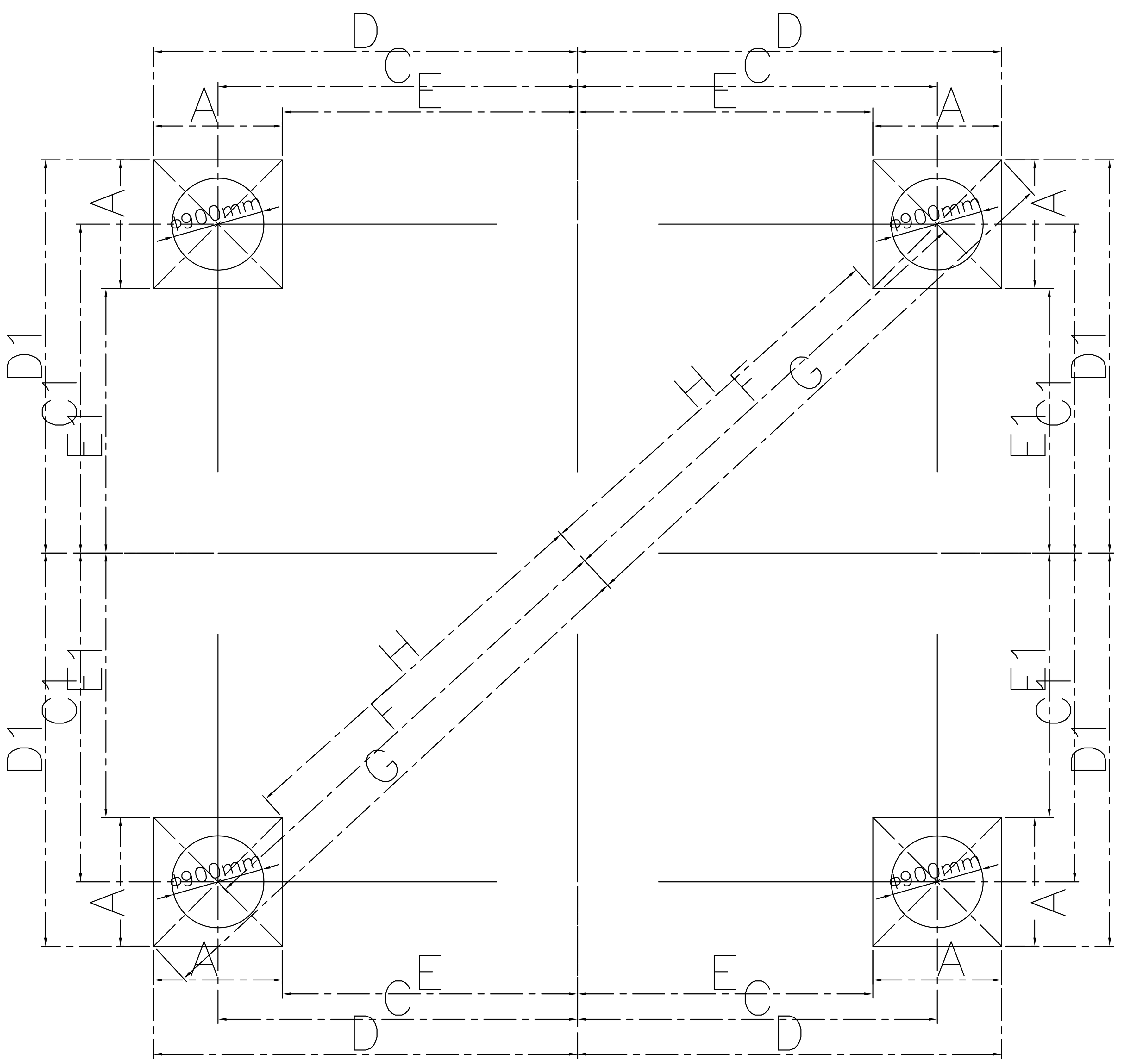
Sheet No. 9 | Total Sh. 13

SHEET NO.: 10 OF 13
DESIGN NO: 295/FDN-P2
TOWER TYPE : TT DA (BB+3mBE)
PILE FOUNDATION LOC. NO.AP104/5

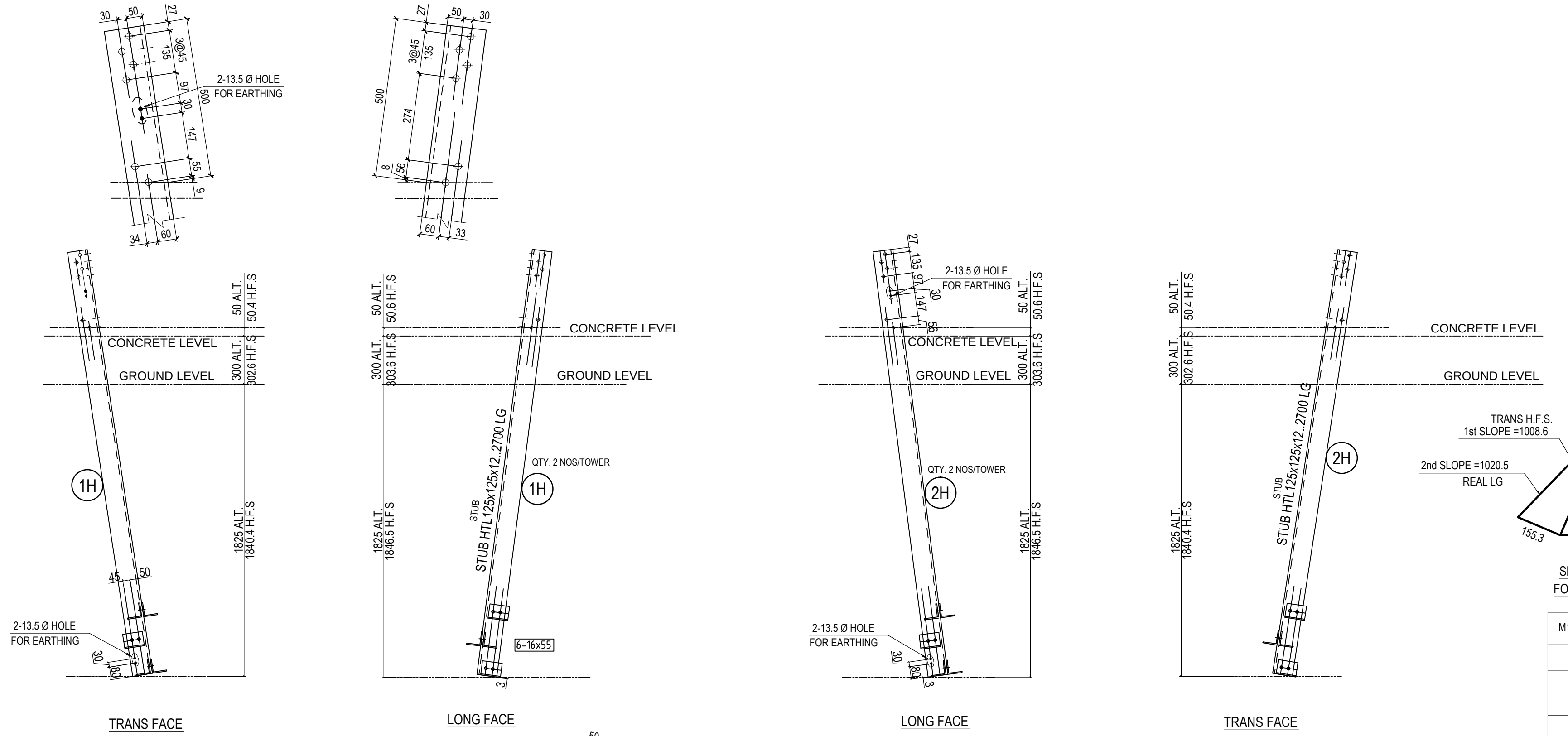


EXCAVATION DIMENSION TABLE											
SOIL TYPE	TOWER TYPE	A	C	C1	D	D1	E	E1	F	G	H
WATER LOGGED (PILE FOUNDATION)	BTB + 3M BE + 0M LE	1200	6111	5440	6711	6040	5511	4840	8182	9029	7335
WATER LOGGED (PILE FOUNDATION)	BTB + 3M BE + 1M LE	1200	6267	5572	6867	6172	5667	4972	8386	9234	7539
WATER LOGGED (PILE FOUNDATION)	BTB + 3M BE + 2M LE	1200	6422	5704	7022	6304	5822	5104	8590	9437	7743
WATER LOGGED (PILE FOUNDATION)	BTB + 3M BE + 3M LE	1200	6577	5835	7177	6435	5977	5235	8793	9640	7946
WATER LOGGED (PILE FOUNDATION)	BTB + 3M BE + 4M LE	1200	6732	5967	7332	6567	6132	5367	8996	9843	8149

B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) FOR TRANSVERSE FACE = 11015 mm
B/B WIDTH AT BTB + 0 M LE (MINIMUM TOWER BASE) FOR LONGITUDINAL FACE = 9868 mm



EXCAVATION DETAIL

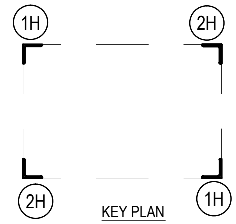


TRANS FACE

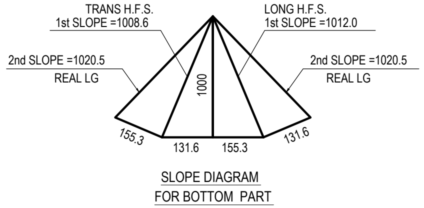
LONG FACE

LONG FACE

TRANS FACE

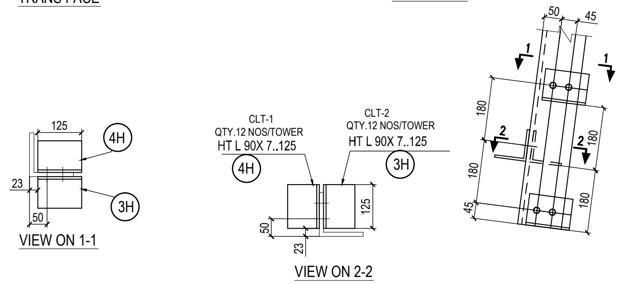


KEY PLAN



SLOPE DIAGRAM
FOR BOTTOM PART

M16 H.R.H. Bolts, Nuts and Washers with 3% extra	
SIZE	QTY./TOWER
55 mm long bolt	25
Spring Washers	25
Std. Flat Washers	25



VIEW ON 1-1

VIEW ON 2-2

NOTES:-

1). All dimensions are in mm.
2). All holes are 17.5 Ø for 16 Ø bolts noted as ○
3). Mild steel shall conform to BSEN 10025 GR-S 235 JO.
4). High tensile steel shall conform to BSEN 10025 GR-S 355 JO.
5). All steel members are to be hot dip galvanized according to ISO 1461 with uniform zinc coating of min.610 g/m^2 & 86µm. thk.
6). Standard spring & flat washers as per DIN 127 (Type B) & DIN 126 respectively to be supplied with each bolt.
7). Min. rolled & cut edge security shall be 22 & 26mm respectively for 16Ø bolts.
8). All item nos are to be prefixed with letter 295DA.
9). All bolts and nuts conforming to ISO 898 (PART 1 & 2) with mechanical properties class 6.8 / 6.
10). All bolts, nuts and washers are to be hot dip galvanized according to ASTM A153.
11). B / N comprises of 1 bolt + 1 nut + 1 spring washer & 1 flat washer.

REFERENCE DRAWINGS	CLIENTS DOCUMENT NO.	REF DRG No.
+0.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5500-K-K-DD-STW-054	295/DA-8
+0.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5500-K-K-DD-STW-055	295/DA-8
+1.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5500-K-K-DD-STW-056	295/DA-9
+1.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5500-K-K-DD-STW-057	295/DA-9
+2.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-058	295/DA-10
+2.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-059	295/DA-10
+3.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-060	295/DA-11
+3.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-061	295/DA-11
+4.0M LEG EXTENSION WITH STEPBOLT	KIP-L-5900-K-K-DD-STW-062	295/DA-12
+4.0M LEG EXTENSION WITHOUT STEPBOLT	KIP-L-5900-K-K-DD-STW-063	295/DA-12
STUB SETTING TEMPLATE	KIP-L-5900-K-K-DD-STW-064	295/DA-2

APPROVAL REFERENCE LETTER No.

L.D.	M.D.	ITEM NO :-
SANDIP 01.12.2016		1 TO 3

Reference Drawings	
Drawing No:	Title:

☐ Approved ☐ Approved with Comments ☐ Not Approved ☐ For Information

Date: ____________ Signature / Stamp

Project Consultant

"Ministry of Energy & Mineral Development
Uganda Electricity Transmission Company Limited

GOPA International Energy Consultants

"CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND
THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP"

Project Drawing: KIP-L-5500-K-K-DD-STW-068-B

	Name:	Date:
Prepared:	HITESH	21.11.2016
Drawn:	SACHIN	21.11.2016
Checked:	N.R.A.	21.11.2016
Approved:	M.B.NENE	21.11.2016

Drawing Detail:
Project:
"Karuma Interconnection
400 KV Karuma - Kawanda Transmission Line"

Revision:		
D		
C		
B	STUB LENGTH REVISED	25.01.2017
A	Ref. No. KIP/L/5500/INK/B-0365	04.01.2017
No.	0	01.12.2016

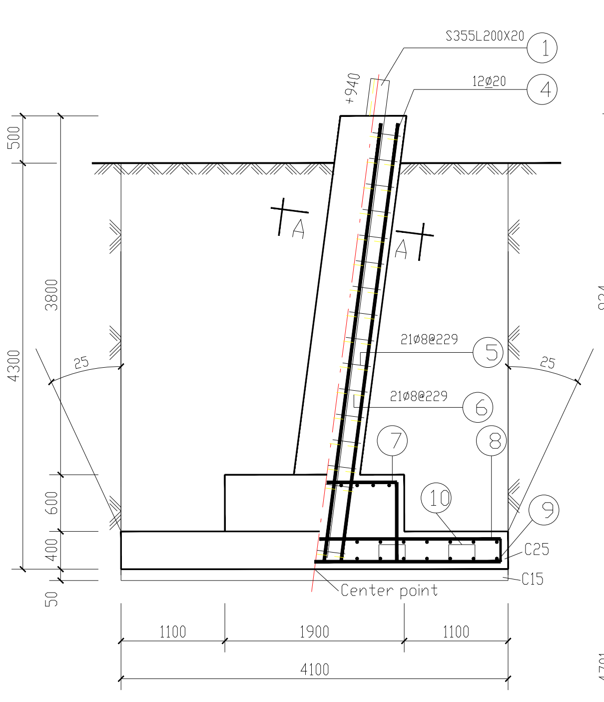
Title:
TOWER TYPE "DA" (0°-2°)
STRUCTURAL DRAWING OF
STUB

Scale: 1 : 25

"Contractor's Drawing No." - - - - - - - - 295/DA-01

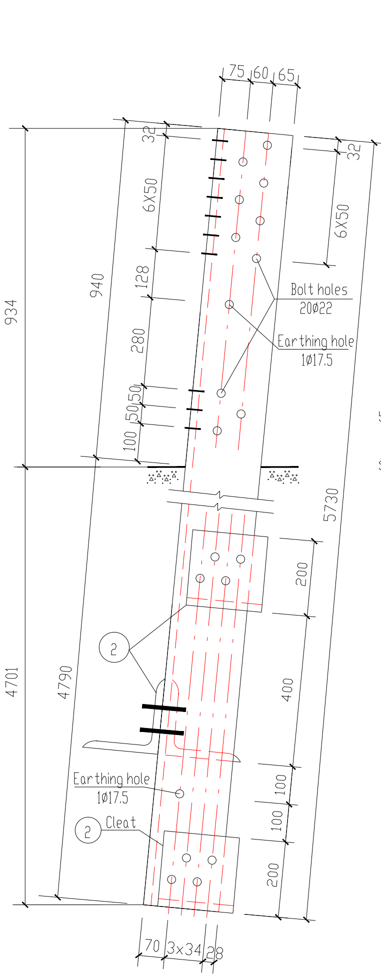
Sheet No.	Total Sh.
1	1

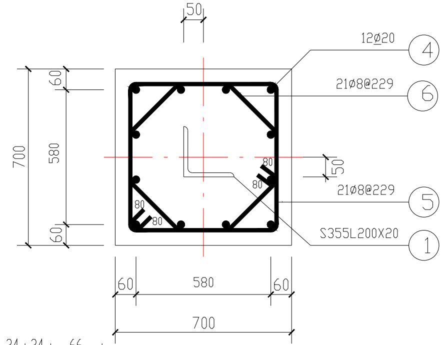
PAPER SIZE A2



Reinforcement drawing

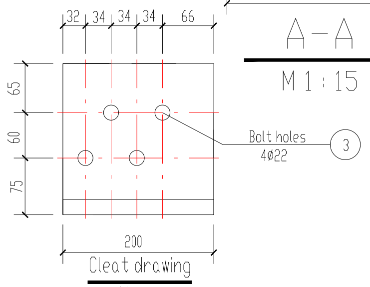
M 1 : 40





A—A

M 1 : 15



Cleat drawing

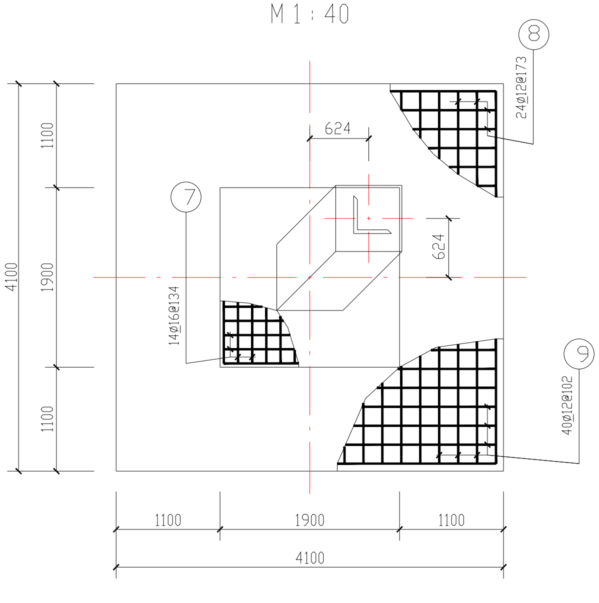
M 1 : 5

Table of materials

NO.	Sort	Spec	Sketch and dimension	Length (mm)	QTY	Units	Weight (kg) PER.PC	Weight (kg) Subtotal
1	Stub angle	S355	L200X20	5730	1	Nos.	344.12	344.1
2	Cleat	S355	L200x20	200	6	Nos.	12.01	72.1
3	Bolts	M20	Grade 6.8	90	12	Nos.	0.40	4.8
4	Main bar	∅20	4780	4780	12	Nos.	11.79	141.5
5	Stirrup	∅8	600 80 80 600	2591	21	Nos.	1.02	21.5
6	Stirrup	∅8	277 208 80	2205	21	Nos.	0.87	18.3
7	Upper slab-main bar	∅16	1800 150 900	3900	28	Nos.	6.16	172.5
8	Bottom slab-top bar	∅12	4000 200	4400	48	Nos.	3.91	187.5
9	Bottom slab-bottom bar	∅12	4000 200	4400	80	Nos.	3.91	312.6
10	Spacer bar	∅8		941	49	Nos.	0.37	18.2
Concrete (m³)	Foundation	C25	1x10.78=10.78				Total steel (kg)	1293.1
	Lean concrete	C15	1x0.84=0.84					

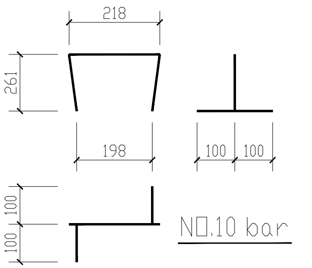
NOTES:

1. All dimensions are in mm, unless otherwise specified.
2. The concrete strength should be no less than 70% of designed strength for erection bar-by-bar, and the concrete strength shall be 100% of designed strength for erection by crane.
3. Minimum cover to main reinforcement shall be 100mm to bottom surface of the lean concrete & 50mm to top of side surface as per specification.
4. ALL Main bars adopt grade 500, and all stirrups adopt grade 250.
5. No foundation shall rest on filled up soil.
6. Not more than 50% bars of the slabs shall be spliced at one section unless specified/shown.
7. The main bars at the corners of the chimneys may not be spliced.
8. ALL hooks, bends, laps, splices& development length shall be as per BS 8110-1:1997 except stated otherwise.
9. The foundation construction should satisfy TSC S2.4 FOUNDATION CONSTRUCTION.
10. ? is the Line Angle.
11. tg a =0.13.

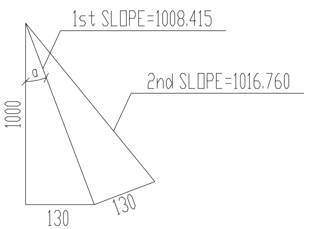


Plane drawing

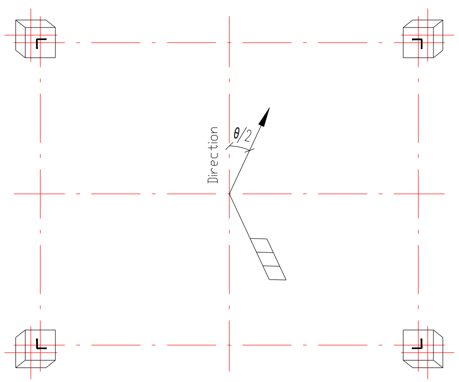
M 1 : 40



NO.10 bar



Slope diagram



Layout plan

Reference Drawings

Drawing No.:	Title:

☐ Approved ☐ Approved with comments ☐ Not Approved ☐ For Information

Date: Signature/Stamp:

Project Consultant

MINISTRY OF ENERGY AND MINERAL DEVELOPMENT
Uganda Electricity Transmission Company Ltd.

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中国电建 POWERCHINA 中国水电 SINOHYDRO

CONSTRUCTION OF THE 600MW KARUMA HYDROPOWER PROJECT AND THE KARUMA INTERCONNECTION PROJECT
MEMD/WORKS/13-14/00017/ERD/EP

Project Drawing: KIP-L-5800-K-L-DD-CFD-006-A

	Name:	Date:
Prepared:		
Drawn:	Yang Zongqi	22/08/2017
Checked:	Zhang Bin	25/08/2017
Approved:	Deng Jinhui	28/08/2017

Drawing Detail:

Project: Karuma-Lira 132kV Transmission Line

Title: Karuma-Lira 132kV TL DD Tower Foundation Class Good Soil

Revision: No.		
D		
C		
B		
A	Yang Zongqi	14/04/2018

Scale: Contractor's Drawing No. Sheet No. 1 Total Sh. 2

Date | Countersign Engineer | Countersign Profession | Date | Countersign Engineer | Countersign Profession | Date | Countersign Engineer | Countersign Profession

Appendix H - Photos

AP 104/5 (PILE LOCATION)- SITE PHOTO SUMMARIES

Pile (900 mm diameter) boring excavation works

Rebar tying and assembly inspections

Pile cap concrete casting

Support piles already cast on site

Figure H-1: AP 104/5 (Pile Location)- Site Photo Summaries

<table>
<tr><th colspan="2">AP 104/5 (PILE LOCATION) – STATIC LOAD TESTS</th></tr>
<tr><td>
Researcher on site assessing the assembly</td><td>
Static Axial Tension/Uplift Load Test</td></tr>
<tr><td>
Lateral Load Foundation Test</td><td>
Static Axial Compression Load Test</td></tr>
</table>

Figure H-2: AP 104/5 (Pile Location) – Static Load Tests

AP 108/15 (ST-POOR SOIL) - SITE PHOTO SUMMARIES

Excavation works ongoing

Rebar assembly and pit side cutting

Sand Sieving works ahead of concrete casting

Preparations for pad and column/chimney concrete casting

Figure H-3: AP 108/15 (ST-Poor Soil) - Site Photo Summaries

AP 108/15 (ST-POOR SOIL) - STATIC LOAD TESTS

Static Axial Tension/Uplift Load Test

Static Axial Tension/Uplift Load Test

Completion Photo - Static Axial Uplift Load Test

Hydraulic Pressure Pump and gauge system

Close-up view of the assemblage by the Researcher

Completion Photo - Static Axial Uplift Load Test

Figure H-4: AP 108/15 (ST-Poor Soil) - Static Load Tests

AP 108/20 (DA-GOOD SOIL) - SITE PHOTO SUMMARIES & STATIC LOAD TESTS	
	
Rebar assembly after completion of excavation works	Level checking on site by the Researcher
	
Concrete casting of Pad and Chimney foundation	Team during joint inspection of Static Axial Tensile Load Test

Figure H-5: AP 108/20 (DA-Good Soil) - Site Photo Summaries & Static Load Tests

KL 30 (B103+5) (DB-WATERLOGGED SOIL) - SITE PHOTO SUMMARIES

Rebar assembly after completion of foundation excavation

De-watering of the Foundation pit

Site QA/QC checks for slump

Concrete Casting for the Step-Pad foundation section

Figure H-6: KL 30 (B103+5) (DB-Waterlogged Soil) - Site Photo Summaries

KL 30 (B103+5) (DB-WATERLOGGED SOIL) – STATIC LOAD TEST

Static Uplift/Tension Load Testing setup

Researcher making test readings

Ground situation during the test at KL 30

Equipment setup for the Test at B103/5 (KL 30)

Commencement of uplift Test at KL 30

Figure H-7: KL 30 (B103+5) (DB-Waterlogged Soil) – Static Load Test

Appendix I - Calculations

FOUNDATION DESIGN CALCULATION FOR AP 108/15

Ultimate Tower Reactions at Base for ST-Tower

Load Case	Joint No.	Load Case No.	Nature of Stub force	Inclined forces on foundation (kN)		
				Compressive/ Uplift	Long side thrust	Transversal side thrust
1	30S	C1- NC NC V_{max}	Compressive	949.65	4.05	1.958
2	30Y	C1- NC NC V_{min}	Uplift	727.20	4.37	0.34
3	30Y	C1- NC NC V_{min}	Uplift	661.76	1.52	3.47
4	30S	C1- NC NC V_{min}	Compressive	855.48	5.86	2.17
Load Case	**Joint No.**	**Load Case No.**	**Nature of Stub force**	**Inclined forces on foundation (kg)**		
				Compressive/ Uplift	**Long side thrust**	**Transversal side thrust**
1	30S	C1- NC NC V_{max}	Compressive	96836	413	200
2	30Y	C1- NC NC V_{min}	Uplift	74153	445	34
3	30Y	C1- NC NC V_{min}	Uplift	67480	155	353
4	30S	C1- NC NC V_{min}	Compressive	87234	598	221
Notes: Above foundation forces are ultimate.						

SOIL PROPERTIES FOR FOUNDATION DESIGN

Please refer schedule4: Design-properties of soil and concrete of technical specification.

S/No.	Soil Type	Poor Soil
1	Assumed mass of Earth for foundations, γ_s (kg/m^3)	1450
2	Assumed mass of rock for foundations, γ_r (kg/m^3)	-
3	Assumed mass of concrete for foundations, γ_c (kg/m^3)	2300
4	Assumed ultimate bearing capacity for foundations under specified maximum ultimate loading, including factor of safety:	
(a)	t/m^2	15
(b)	kN/m^2	147.099
5	Ultimate shear stress in rock, τ	
(a)	t/m^2	-
(b)	kN/m^2	-
6	Assumed angle to vertical of frustum of earth resisting uplift (angle of Repose).	15°
7	Assumed angle to vertical of frustum of earth resisting uplift (angle of repose) considered in foundation design, Ø.	12°

Client: Samuel Acidri/KPTL	**FOUNDATION DESIGN CALCULATION FOR AP 108/15**			
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
1	Leg-angle, Ø	Degrees	Ø	10.942
2	Length factor			1.037
3	Chimney/Column			Inclined
4	**Maximum Loads as per FAT Test**			
a)	P-compression, P_{comp}	kN	P_{comp}	949.65
b)	P-tension, $P_{tension}$	kN	$P_{tension}$	727.20
c)	Shear -transversal, Tr	kN	Tr	3.47
c)	Shear -longitudinal, Lg	kN	Lg	5.86
5	Footing type			Double slab
6	**Footing Dimensions**			
a)	Footing depth, H	m	H	2.75
b)	Footing dimensions, L x W x H	m	$L\ x\ W\ x\ H$	4.39 x 4.39 x 0.35
c)	Step, L x W x H	m	$L\ x\ W\ x\ H$	1.88 x 1.88 x 0.275
d)	Footing base, B	m	B	4.39
e)	Lean pad height, P	m	P	0.05
7	**Inclined Column/Chimney Dimensions**			
	Size, W	m	W	0.6 x 0.6
	Muff, M	mm	M	800
	Chimney height, H_1	mm	H_1	2075
	Base thickness, D	mm	D	350
	Step depth, D_1	mm	D_1	275
	Step width, B_1	mm	B_1	1880
	Working point height, Wp	mm	W_p	625
8	**Footing Reinforcement**			
	Bottom	No. -Dia		29 No, Dia 14 @ 153 c/c/
	Top -step	No. -Dia		10 No, Dia 14 @ 197 c/c
	Top -base	No. -Dia		16 No, Dia 14 @ 286 c/c
9	**Column/Chimney reinforcement**			
	Main internal rebars	No. -Dia		8No, Dia 20 @ 153 c/c/
	Main corner rebars	No. -Dia		4 No, Dia 20 @ 197 c/c
	Links	No. -Dia		Dia 8 @ 225 c/c
10	**Material Data**			
	Concrete density in RCC and PCC (dry), γ_c	kg/m^3	γ_c	2300
	Concrete cover to bottom surface, c	mm	c	100
	Concrete cover to top and side surfaces, c'	mm	c'	50
	Characteristic concrete strength, f_{ck}	MPa	f_{ck}	C25
	Characteristic steel strength, f_{yk}	MPa	f_{yk}	f_{yk} 500
11	**Data for Checks (Soil cone from edge)**			
	Height of soil cone, H_b	m	H_b	2.05
	Effective soil weight (soil cone + excavation pit weights)	kN	W_s	679.501
	Effective concrete weight, W_c	kN	W_c	77.051
	-in muff		$\gamma_c\ x\ M\ x\ W^2 = \left[\left(\frac{2300x9.81}{1000}\right)\ x\ 0.8\ x\ 0.6^2\right]$	6.496
	-in soil		$\left[(H_1 x W^2) + \left((D\ x\ B^2) + (D_1 x {B_1}^2)\right)\right] x(\gamma_c - \gamma_s)$ $(H_1 x W^2) = (2.075\ x\ 0.6^2) = 0.747$ $(D\ x\ B^2) = (0.35\ x\ 4.39^2) = 6.745235$ $(D_1 x {B_1}^2) = (0.275\ x\ 1.88^2) = 0.97196$ $\gamma_c = \left(\frac{2300\ x\ 9.81}{1000}\right) = 22.563\ \&\ \gamma_s = \left(\frac{1450\ x\ 9.81}{1000}\right) = 14.2245$ $\Rightarrow [0.747 + (6.745235 + 0.97196)] x\ (22.563 - 14.2245)$	70.555
12	**Uplift check**			
	Load case	No.		2

S/No.	Description	Unit	Calculations/Equations	Results/Remarks
Client: Samuel Acidri/KPTL			**FOUNDATION DESIGN CALCULATION FOR AP 108/15**	
	Load	kN		727.2
	Factor of safety (FoS)			1.040
13	**Overturning check**			
	Load case	No.		2
	Tensile load (Uplift Load)	kN	Uplift	727.2
	Shear force- transversal, (Tr)	kN	Tr	0.34
	Shear force- longitudinal, (Lg)	kN	Lg	4.37
	Moment due to uplift	kNm	$M_{Uplift} = Uplift\ x\ \cos^2\theta\ x\ \frac{B}{3}$ $\Rightarrow M_{Uplift} = \left[727.2\ x\ (\cos 12)^2\ x\ \left(\frac{4.39}{3}\right)\right]$	1018.136
	Moment due to side thrust	kNm	$M_{ST} = Max\ (Tr, Lg, ST)\ x\ (M + H - P)$ $\Rightarrow M_{ST} = 4.37\ x\ (0.8 + 2.75 - 0.05)$	15.295
	Moment due to concrete	kNm	$M_c = \left(W_c\ x \frac{B}{3}\right) = \left(77.051\ \text{x}\ \frac{4.39}{3}\right)$	112.751
	Resisting moment due to soil	kNm	$M_s = \left[\left(\frac{W_s}{2}\right)\ x\ \left(\frac{5}{6}\right) x\ B\right] = \left[\left(\frac{679.501}{2}\right)\ x\ \left(\frac{5}{6}\right) x\ 4.39\right]$	1242.921
	Total overturning moment	kNm	$M_T = M_{Uplift} + M_{ST} - M_c = (1018.136 + 15.295 - 112.751)$ $\Rightarrow Since\ M_T < M_s, then\ its\ OK.$	920.680
	Factor of safety, FoS		$FoS = \frac{M_s}{M_T} = \left(\frac{1242.921}{920.680}\right)$	1.350
14	**Base pressure check**			
	Load case	No.		1
	Compressive load	kN		949.65
	Shear force (Tr)- transversal force	kN	T.F	1.958
	Shear force (Lg)- longitudinal force	kN	L.F	4.05
	Balancing height for shear in transversal direction (Tr) -B_{Ht}	m	$B_{Ht\ (Tr)} = \frac{Shear\ force\ (Tr)}{\sqrt{Passive\ pressure}}$ NOTE: Balancing height is greater than chimney/column height. Therefore, it is restricted to the chimney/column height. Passive pressure $(\mathrm{P_p}) = 0.5\ \mathrm{x}\ \mathrm{k_p}\ \mathrm{x}\ \gamma_s\ \mathrm{xW}$ Where: $k_p = (1 + \sin\emptyset)/(1 - \sin\emptyset)$ $\gamma_s = \left(\frac{1450\ x\ 9.81}{1000}\right) = 14.2245$	0.549
	Moment due to passive transversal force (Tr)- $M_{p\ (Tr)}$	kNm	$M_{passive}$ (Tr) $M_{p\ (Tr)} = P_p\ x\ (B_{Ht})^2\ x\ \left\{\left(\frac{B_{Ht}}{3}\right) + (H_1 - 0.3 - B_{Ht}) + D + D_1\right\}$	3.983
	Balancing height for shear in longitudinal direction (Lg)	m	$B_{Ht\ (Lg)} = \frac{Shear\ force\ (Lg)}{\sqrt{Passive\ pressure}}$ Passive pressure $(\mathrm{P_p}) = 0.5\ \mathrm{x}\ \mathrm{k_p}\ \mathrm{x}\ \gamma_s\ \mathrm{xW}$ Where: $k_p = (1 + \sin\emptyset)/(1 - \sin\emptyset)$	0.789
	Moment due to passive longitudinal force (Lg)- $M_{p\ (Lg)}$	kNm	$M_{passive}$ (Lg) $M_{p\ (Lg)} = P_p\ x\ (B_{Ht})^2\ x\ \left\{\left(\frac{B_{Ht}}{3}\right) + (H_1 - 0.3 - B_{Ht}) + D + D_1\right\}$	7.59
	Soil bearing capacity available:	kN/m^2		147.099
	= P/A	kN/m^2	$\frac{P}{A} = \frac{\left(\frac{P_{comp}}{L.F}\ x\ Overload\right)}{B^2}$	51.915
	= P_{ex}/Z	kN/m^2	$\frac{P_{ex}}{Z} = 2\ x\ \left(\frac{P_{comp}}{L.F}\right) x\ \tan\theta\ x\ W_p\ x\ \frac{6}{B^3}$	15.689
	= M/Z (Transversal), $M_{(Tr)}$	kN/m^2	$\{M_{(Tr)} - M_{p\ (Tr)}\}\ x\ \frac{6}{B^3}$ $M_{(Tr)} = Shear\ force\ (Tr) x\ Lever\ arm$ Where: $Lever\ arm = H + M - P$	0.204
	= M/Z (Longitudinal), $M_{(Lg)}$	kN/m^2	$\{M_{(Lg)} - M_{p\ (Lg)}\}\ x\ \frac{6}{B^3}$ $M_{(Lg)} = Shear\ force\ (Lg) x\ Lever\ arm$ Where: $Lever\ arm = H + M - P$	0.467
	= Maximum pressure	kN/m^2	$\frac{P}{A} + \frac{P_{ex}}{Z} + M_{(Tr)} + M_{(Lg)}$	68.275
	= Minimum pressure	kN/m^2	$\frac{P}{A} - \frac{P_{ex}}{Z} - M_{(Tr)} - M_{(Lg)}$	35.555
	Factor of safety (FoS)		$FoS = \frac{Soil\ bearing\ capacity}{Max. Pressure} = \frac{147.099}{68.275}$	2.155

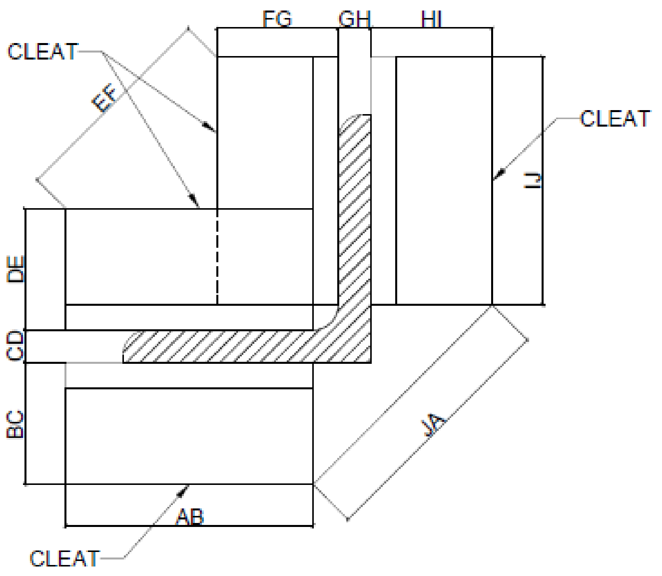
Client: Samuel Acidri/KPTL	**FOUNDATION DESIGN CALCULATION FOR AP 108/15**			
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
15	**Design for Bending for Bottom**			
	Load case	No.		1
	Compressive load	kN		949.65
	Shear force- transversal (Tr)	kN		1.958
	Shear force- longitudinal (Lg)	kN		4.05
	Maximum pressure, P_1	kN/m^2	$P_1 = \frac{P}{A} + \frac{P_{ex}}{Z} + Max\left[M_{(Tr)}, M_{(Lg)}\right]$	59.963
	Bending at face of chimney			
	Effective depth		$d_t = \left[(D + D_1) - C_{cover\,(bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$ $\Rightarrow d_t = \left[(350 + 275) - 100 - 14 - \frac{14}{2}\right]$	504
	Bending Moment -design, BM_{Des}	kNm	$BM_{Des} = \left[P_1\ x\ \frac{(B - W)^2}{8}\ x\ B\right]$ $\Rightarrow BM_{Des} = \left[59.963\ x\ \frac{(4.39 - 0.6)^2}{8}\ x\ 4.39\right]$	472.647
	Area of steel required, $A_{st}(Req)$	mm^2	$A_{st}(Req) = \frac{M}{0.87\ f_{yk}\ Z}$ $k = \frac{M}{bd^2\ f_{ck}} = \frac{472.647\ x\ 10^6}{4390\ x\ 504^2\ x\ 25} = 0.017 \leq 0.15, Hence\ OK.$ $Z = d\left[0.5 + \left(0.25 - \frac{k}{0.9}\right)^{0.5}\right] \ngtr 0.95d$ $\Longrightarrow Z = 504\left[0.5 + \left(0.25 - \frac{0.017}{0.9}\right)^{0.5}\right] = 494.293mm$ $0.95d = 0.95\ x\ 504 = 478.80mm$ $\Longrightarrow Z = 494.293mm \ngtr 0.95d = 478.80mm,$ $Hence\ take\ Z = 0.95d = 478.80mm, OK.$ $\Longrightarrow A_{st}(Req) = \frac{M}{0.87\ f_{yk}\ Z} = \frac{472.647\ x\ 10^6}{0.87\ x\ 500\ x\ 478.80}$	2269.308
	Area of steel provided, $A_{st}(Prov)$	mm^2	$A_{st}(Prov) = \left(\frac{\pi D^2}{4} x\ No\ of\ bars\right) = \left(\frac{\pi\ x\ 14^2}{4} x\ 29\right)$ $Since\ A_{st\,(Req)} < A_{st\,(prov)}, Hence\ OK.$	4464.203
	Bending Moment -resisting, BM_{Res}	kNm	$BM_{Res} = 0.87 f_{yk}\ x\ A_{st}\ x\ d_t\ \left(1 - \frac{0.9738\ x\ f_{yk}\ x\ A_{st}}{f_{ck}\ x\ B_1\ x\ d_t}\right)$ $\Longrightarrow 0.87 f_{yk}\ x\ A_{st\,(prov)}\ x\ d_t\ \left(1 - \frac{0.9738\ x\ f_{yk}\ x\ A_{st\,(prov)}}{f_{ck}\ x\ B_1\ x\ d_t}\right)$ $= \left[0.87\ x\ 500\ x\ 4464.203\ x\ 504\ x\left(1 - \frac{0.9738\ x\ 500\ x\ 4464.203}{25\ x\ 1880\ x\ 504}\right)\right] x 10^{-6}$ $Since, BM_{Res} = 888.923\ kNm > BM_{Des} = 472.647\ kNm, Hence\ OK.$	888.923
	Factor of safety (FoS)		$FoS = \frac{BM_{Res}}{BM_{Des}} = \left(\frac{888.923}{472.647}\right)$	1.88
	Bending at face of step			
	Effective depth	mm	$d_b = \left[(D + D_1) - C_{cover\,(bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$ $\Rightarrow d_b = \left[(350) - 100 - 14 - \frac{14}{2}\right]$	229
	Bending Moment -design, BM_{Des}	kNm	$BM_{Des} = \left[P_1\ x\ \frac{(B - B_1)^2}{8}\ x\ B\right]$ $\Rightarrow BM_{Des} = \left[59.963\ x\ \frac{(4.39 - 1.88)^2}{8}\ x\ 4.39\right]$	207.303
	Area of steel required, $A_{st}(Req)$	mm^2	$A_{st}(Req)$	2172.476
	Area of steel provided, $A_{st}(Prov)$	mm^2	$A_{st}(Prov) = \left(\frac{\pi D^2}{4} x\ No\ of\ bars\right) = \left(\frac{\pi\ x\ 14^2}{4} x\ 29\right)$	4464.203
	Bending Moment -resisting, BM_{Res}	kNm	$\left[0.87 f_{yk}\ x\ A_{st\,(prov)}\ x\ d_b\left(1 - \frac{0.9738\ f_{yk}\ x\ A_{st\,(prov)}}{f_{ck}\ x\ B\ x\ d_b}\right)\right] x 10^{-6}$ $Since, BM_{Res} = 406.241\ kNm > BM_{Des} = 207.303\ kNm, Hence\ OK.$	406.241
	Factor of safety (FoS)		$FoS = \frac{BM_{Res}}{BM_{Des}} = \left(\frac{406.241}{207.303}\right)$	1.96
	Percentage of steel provided in the footing, P_T	%	$P_T = 100\ x\ \frac{A_{st}(Prov)}{Total\ Cross\ sectional\ Area}$ $\Longrightarrow P_T = 100\ x\ \left[\frac{4464.203}{(4390\ x\ 350) + (1880\ x\ 275)}\right]$	0.217
16	**Design for Bending for Top**			
	Load case	No.		2
	Compressive load	kN		727.2

Client: Samuel Acidri/KPTL			**FOUNDATION DESIGN CALCULATION FOR AP 108/15**	
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
	Shear force- transversal (Tr)	kN		0.34
	Shear force- longitudinal (Lg)	kN		4.37
	Maximum pressure, P_2	kN/m^2	$P_2 = \frac{Uplift}{(B^2 - W^2)} = \left[\frac{727.2}{(4.39^2 - 0.6^2)}\right]$	38.452
	Bending at face of chimney			
	Bending Moment -design, BM_{Des}	kNm	$BM_{Des} = \left[P_2\ x\ \frac{(B-W)^2}{8}\ x\ B\right]$ $\Rightarrow BM_{Des} = \left[38.452\ x\ \frac{(4.39-0.6)^2}{8}\ x\ 4.39\right]$	303.087
	Area of steel required, $A_{st}(Req)$	mm^2	$A_{st}(Req)$ $k = \frac{M}{bd^2\ f_{ck}} = \frac{303.087\ x\ 10^6}{4390\ x\ 504^2\ x\ 25} = 0.011 \leq 0.15, Hence\ OK.$ $Z = d\left[0.5 + \left(0.25 - \frac{k}{0.9}\right)^{0.5}\right] \ngtr 0.95d$ $\Longrightarrow Z = 504\left[0.5 + \left(0.25 - \frac{0.011}{0.9}\right)^{0.5}\right] = 497.763mm$ $0.95d = 0.95\ x\ 504 = 478.80mm$ $\Longrightarrow Z = 497.763mm \ngtr 0.95d = 478.80mm$ $Hence\ take\ Z = 0.95d = 478.80mm, OK.$ $\Longrightarrow A_{st}(Req) = \frac{M}{0.87\ f_{yk}\ Z} = \frac{303.087\ x\ 10^6}{0.87\ x\ 500\ x\ 478.80}$	1455.204
	Area of steel provided, $A_{st}(Prov)$	mm^2	$A_{st}(Prov) = \left(\frac{\pi D^2}{4} x\ No\ of\ bars\right) = \left(\frac{\pi\ x\ 14^2}{4} x\ 10\right)$	1539.38
	Bending Moment -resisting, BM_{Res}	kNm	$\left[0.87 f_{yk}\ x\ A_{st\ (prov)}\ x\ d_t\left(1 - \frac{0.9738\ f_{yk}\ x\ A_{st\ (prov)}}{f_{ck}\ x\ B_1\ x\ d_t}\right)\right] x 10^{-6}$ $= \left[0.87\ x\ 500\ x\ 1539.38\ x\ 504\ x\left(1 - \frac{0.9738\ x\ 500\ x\ 1539.38}{25\ x\ 1880\ x\ 504}\right)\right] x 10^{-6}$	337.472
	Factor of safety (FoS)		$FoS = \frac{BM_{Res}}{BM_{Des}} = \left(\frac{337.472}{303.087}\right)$	1.114
	Bending at face of step			
	Bending Moment -design, BM_{Des}	kNm	$BM_{Des} = \left[P_2\ x\ \frac{(B-B_1)^2}{8}\ x\ B\right]$ $\Rightarrow BM_{Des} = \left[38.452\ x\ \frac{(4.39-1.880)^2}{8}\ x\ 4.39\right]$	132.934
	Area of steel required, $A_{st}(Req)$	mm^2	$A_{st}(Req)$ $k = \frac{M}{bd^2\ f_{ck}} = \frac{132.934\ x\ 10^6}{1880\ x\ 275^2\ x\ 25} = 0.0374 \leq 0.15, Hence\ OK.$ $Z = d\left[0.5 + \left(0.25 - \frac{0.0374}{0.9}\right)^{0.5}\right] \ngtr 0.95d$ $\Longrightarrow Z = 504\left[0.5 + \left(0.25 - \frac{0.0374}{0.9}\right)^{0.5}\right] = 482.105mm$ $\Longrightarrow 0.95d = 0.95\ x\ 504 = 478.80mm$ $\Longrightarrow Z = 482.105mm \ngtr 0.95d = 478.80mm, Hence\ take\ Z = 0.95d$ $\Longrightarrow A_{st}(Req) = \frac{M}{0.87\ f_{yk}\ Z} = \frac{132.934\ x\ 10^6}{0.87\ x\ 500\ x\ 478.80}$	1370.886
	Area of steel provided, $A_{st}(Prov)$	mm^2	$A_{st}(Prov) = \left(\frac{\pi D^2}{4} x\ No\ of\ bars\right) = \left(\frac{\pi\ x\ 14^2}{4} x\ 16\right)$	2463.01
	Bending Moment -resisting, BM_{Res}	kNm	$\left[0.87 f_{yk}\ x\ A_{st\ (prov)}\ x\ d_b\left(1 - \frac{0.9738\ f_{yk}\ x\ A_{st\ (prov)}}{f_{ck}\ x\ B\ x\ d_b}\right)\right] x 10^{-6}$ $= \left[0.87\ x\ 500\ x\ 2463.01\ x\ 229 x\left(1 - \frac{0.9738\ x\ 500\ x\ 2463.01}{25\ x\ 4390\ x\ 229}\right)\right] x 10^{-6}$	233.645
	Factor of safety (FoS)		$FoS = \frac{BM_{Res}}{BM_{Des}} = \left(\frac{233.645}{132.934}\right)$	1.758
	Percentage of steel provided in the footing, P_T	%	$P_T = 100\ x\ \frac{A_{st\ (Prov)}}{Total\ Cross\ sectional\ Area}$ $\Longrightarrow P_T = 100\ x\ \left[\frac{(1539.38 + 2463.01)}{(4390\ x\ 350) + (1880\ x\ 275)}\right]$	0.195
17	**Design for Shear**			
	Load case	No.		1
	Compressive load	kN		949.65
	Shear force- transversal (Tr)	kN		1.958

Client: Samuel Acidri/KPTL			**FOUNDATION DESIGN CALCULATION FOR AP 108/15**	
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
	Shear force- longitudinal (Lg)	kN		4.05
	Maximum pressure, P_1	kN/m^2	P_1	59.963
	Shear-check at face of chimney (Compressive)			
	Shear force-compressive, V_{comp}	kN		1134.027
	Resisting depth, d_{Res}	mm	$d_{Res} = \left[(D + D_1) - C_{cover\,(bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$ $\Rightarrow d_t = \left[(350 + 275) - 100 - 14 - \frac{14}{2}\right]$	504
	Resisting Area, A_{Res}	mm^2	$A_{Res} = LxW = 504\ x\ 2400$	1209600
	Shear stress- Actual, V_{act}	N/mm^2		0.938
	Shear stress- permissible, V_{perm}	N/mm^2	$\min\left(0.8\ x\ \sqrt{f_{ck}}\right), 5$	4
	Factor of safety (FoS)			4.267
	Shear-check at *1.5d'* from face of chimney			
	Shear force-compressive, V_{comp}	kN		888.146
	Resisting depth, d	mm	$d_b = \left[(D + D_1) - C_{cover\,(bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$ $\Rightarrow d_b = \left[(350) - 100 - 14 - \frac{14}{2}\right]$	229
	Resisting Area, A	mm^2		1934592
	Percentage of steel provided in the footing, P_T	%	$P_T = 100\ x\ \frac{A_{st\,(prov)}}{Total\ Cross\ sectional\ Area}$	0.444
	Shear stress- Actual, V_{act}	N/mm^2		0.459
	Shear stress- permissible, V_{perm}	N/mm^2	$\left[\frac{0.79\ x\ (P_T)^{\frac{1}{3}}\ x\ \left(\frac{400}{d}\right)^{\frac{1}{4}}}{1.25}\right] x\ f_{ck\,(factor)}$	0.554
	Factor of safety (FoS)			1.207
	Shear-check at face of Step			
	Shear force	kN		943.680
	Resisting depth, d	mm	$d = \left[(D + D_1) - C_{cover\,(bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$	229
	Resisting Area, A	mm^2		1722080
	Shear stress- Actual, V_{act}	N/mm^2		0.548
	Shear stress- permissible, V_{perm}	N/mm^2	$\min\left(0.8\ x\ \sqrt{f_{ck}}\right), 5$	4
	Factor of safety (FoS)			7.299
	Shear-check at *d'* from face of Step			
	Shear force, V	kN		270.082
	Resisting depth, d	mm	$d_b = \left[(D + D_1) - C_{cover\,(bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$ $\Rightarrow d_b = \left[(350) - 100 - 14 - \frac{14}{2}\right]$	229
	Resisting Area, A	mm^2		1005310
	Percentage of steel provided in the footing, P_T	%	$P_T = 100\ x\ \frac{A_{st\,(prov)}}{Total\ Cross\ sectional\ Area}$	0.444
	Shear stress- Actual, V_{act}	N/mm^2		0.269
	Shear stress- permissible, V_{perm}	N/mm^2	$\left[\frac{0.79\ x\ (P_T)^{\frac{1}{3}}\ x\ \left(\frac{400}{d}\right)^{\frac{1}{4}}}{1.25}\right] x\ f_{ck\,(factor)}$	0.554
	Factor of safety (FoS)			2.063
	Shear-check at *1.5d'* from face of Step/Haunch			
	Shear force, V	kN		760.488
	Resisting depth, d	mm		229
	Resisting Area, A	mm^2		2351372
	Percentage of steel provided in the footing, P_T	%	$P_T = 100\ x\ \frac{A_{st\,(prov)}}{Total\ Cross\ sectional\ Area}$	0.444
	Shear stress- Actual, V_{act}	N/mm^2		0.323
	Shear stress- permissible, V_{perm}	N/mm^2	$\left[\frac{0.79\ x\ (P_T)^{\frac{1}{3}}\ x\ \left(\frac{400}{d}\right)^{\frac{1}{4}}}{1.25}\right] x\ f_{ck\,(factor)}$	0.554
	Factor of safety (FoS)			1.714
18	**Chimney/Inclined Column design- Main reinforcement**			
	Load case	No.		1

Client: Samuel Acidri/KPTL	**FOUNDATION DESIGN CALCULATION FOR AP 108/15**			
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
	Axial Load	kN		949.65
	Shear force- Transversal (Tr)	kN		1.958
	Shear force- Longitudinal (Lg)	kN		4.05
	Uplift Load (design), $P_{u\,(design)}$	kN	$P_{u\,(design)}$	949.65
	Lever Arm	m	$Lever\ arm = \frac{(H_1 + M)}{1000}$	2.875
	Moment due to Transversal shear force (Tr), $M_{(Tr)}$	kNm	$M_{(Tr)} = Shear\ force\ (Tr) x\ Lever\ arm$	5.629
	Moment due to Longitudinal shear force (Lg), $M_{(Lg)}$	kNm	$M_{(Lg)} = Shear\ force\ (Lg) x\ Lever\ arm$	11.644
	Moment due to passive transversal pressure (Tr), $M_{(pr-Tr)}$	kNm	$M_{(pr-Tr)} = Passive\ force\ x\ Lever\ arm$ Where: $Passive\ pressure\ (P_p) = 0.5\ x\ k_p\ x\ \gamma_s\ x\ W x\ (B_{Ht})^2$ $Lever\ Arm = \frac{B_{Ht}}{3} + \ (H_1 - 0.3 - B_{Ht})$	2.759
	Moment due to passive Longitudinal pressure (Lg), $M_{(pr-Lg)}$	kNm	$M_{(pr-Lg)} = Passive\ force\ x\ Lever\ arm$ Where: $Passive\ pressure\ (P_p) = 0.5\ x\ k_p\ x\ \gamma_s\ x\ W x\ (B_{Ht})^2$ $Lever\ Arm = \frac{B_{Ht}}{3} + \ (H_1 - 0.3 - B_{Ht})$	5.058
	Uplift Moment- design (transversal), $M_{u\,(Tr,\ design)}$	kNm	$M_{u\,(Tr,\ design)} = M_{(Tr)} - M_{(pr-Tr)}$	2.87
	Uplift Moment- design (longitudinal), $M_{u\,(Lg,\ design)}$	kNm	$M_{u\,(Lg,\ design)} = M_{(Lg)} - M_{(pr-Lg)}$	6.586
	Uplift Moment- uniaxially converted, $M_{u\,(u,conv)}$	kNm	$M_{u\,(u,conv)} = M_{u\,(Tr,\ design)} + M_{u\,(Lg,\ design)}$	9.456
	Uplift Moment- uniaxial capacity, $M_{u\,(u,cap)}$	kNm	PM Curve	534.6
	Factor of Safety (FoS)		$FoS = \frac{M_{u\,(u,cap)}}{M_{u\,(u,conv)}} = \frac{534.6}{9.456}$	56.536
	Percentage of steel provided in chimney/inclined column	%	$P_T = 100\ x\ \frac{A_{st\,(prov)}}{Total\ Cross\ sectional\ Area}$ $\Rightarrow P_T = 100\ x\ \left[\frac{(2513.274 + 1256.637)}{(600\ x\ 600)}\right]$	1.047

STUB DESIGN FOR AP 108/15				
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
1	**Tower Loads (Ultimate) for maximum body extension**			
a)	Compressive weight	kg		96836
	Compressive Load	kN	$F = mg = \left[\frac{(96836\ x\ 9.81)}{1000}\right]$	949.96
	Side thrust (transversal)	kN		3.46
	Side thrust (Longitudinal)	kN		1.52
b)	Tensile weight	kg		74128
	Tensile Load	kN	$F = mg = \left[\frac{(74128\ x\ 9.81)}{1000}\right]$	727.20
	Side thrust (transversal)	kN		3.46
	Side thrust (Longitudinal)	kN		1.52
2	**Stub Details**			
	Stub arrangement	No.		1 single angle
	Stub material			High Tensile (HT)
	Stub section	mm		200 x 200 x 18
	Stub depth inside concrete	mm		2243
3	**Cleat Details**			
	Cleat arrangement	No.		2 (back to back)
	Cleat material			HT
	Cleat section	mm		125 x 125 x 12
	Cleat 1 length	mm	L_1	170
	Cleat 1 number	No.		3

	Cleat 2 length	mm	L_2	170
	Cleat 2 number	No.		3
4	**Bolt Details**			
	Diameter of bolts	mm		16
	No. of bolts	No.		3
5	**Concrete Details**			
	Grade of concrete, f_{ck}	N/mm^2	f_{ck}	25
6	**Stub Area Check as per ASCE 52**			
	Stub flange width	mm		200
	Stub thickness	mm		18
	Stub material, f_{yk}	kg/cm^2		3518
	Stub area provided	cm^2		69.301
	Compressive weight	kg	P	96836
	Transversal side thickness weight	kg		353
	Longitudinal side thickness weight	kg		155
	Resistance side thickness weight	kg	V	385
	Stub area required	cm^2		27.67
	Tensile weight	kg	P	74128
	Transversal side thickness weight	kg		353
	Longitudinal side thickness weight	kg		155
	Resistance side thickness weight	kg	V	385
	Stub area required	cm^2		22.22
7	**Check for stub length in compression** 			
	Depth of stub in concrete	cm	$D_{stub\ (concrete)}$	224.30
	Punching shear stress for compression	N/mm^2	$[0.35\ x\ \sqrt{f_{ck}}] = [0.35\ x\ \sqrt{25}]$	1.75
		kg/cm^2	$V_{shear\ (comp)} = [0.35\ x\ \sqrt{f_{ck}}] = \left[0.35\ x\ \sqrt{\left(\frac{25}{9.81\ x (\ 0.1)^2}\right)}\right]$	17.838
	Perimeter for shear	cm	$P = AB + BC + CD + DE + EF + FG + GH + HI + IJ + JA$ $P = 170 + 125 + 18 + 125 + 90 + 125 + 18 + 125 + 170 + 228$	119.4
	Therefore, punching shear strength	kg	$V_{shear\ (strength)} = (D_{stub\ (conc)}\ x\ P\ x\ V_{shear\ (comp)}) > Compressive\ wt$ $Compressive\ weight = 96836\ kg$ $\Longrightarrow V_{shear\ (strength)} = 224.30\ x\ 119.40\ x\ 17.838$ $\Longrightarrow V_{shear\ (strength)} = 477727\ kg > 96836\ kg$	477727
	Factor of Safety, FoS		$FoS = \frac{477727\ kg}{96836\ kg} = 4.94 > 1, Hence, SAFE$	4.94
8	**Check for Stub Length in Uplift**			
	Punching shear stress for uplift	kg/cm^2	$V_{shear\ (uplift)} = [0.28\ x\ \sqrt{f_{ck}}] = [0.28\ x\ \sqrt{25}]$	14.271
	Therefore, punching shear strength	kg	$V_{shear\ (strength)} = (D_{stub\ (conc)}\ x\ P\ x\ V_{shear\ (uplift)}) > Tensile\ wt$ $Tensile\ weight = 74128\ kg$ $\Longrightarrow V_{shear\ (strength)} = 224.30\ x\ 119.40\ x\ 14.271$ $\Longrightarrow V_{shear\ (strength)} = 382198\ kg > 74128\ kg$	382198
	Factor of Safety, FoS		$FoS = 382198\ kg/74128\ \ kg = 5.16 > 1, Hence, SAFE$	5.16

S/No.	Description	Unit	Calculations/Equations	Results/Remarks
	CLEAT DESIGN AS PER ASCE 52 As per appendix 8 of the Technical Specification, Schedule 4: The design properties of the soil and concrete are:			
1	Ultimate Compression for Cleat design (50% Load)			
	Ultimate compression weight	kg	Weight	48418
	Ultimate compression load	kN	$F = mg = \left[\frac{(48418\ x\ 9.81)}{1000}\right]$	474.981
2	Concrete grade	N/mm^2	f_{ck}	25
		N/mm^2	$f_{ck}{}' = (0.8\ x\ f_{ck}) = 0.8\ x\ 25$	20
		kg/cm^2	$f_{ck}{}' = (0.8\ x\ f_{ck}) = \left[0.8\ x\ \frac{25}{9.81\ x\ (0.1)^2}\right]$	203.9
3	Cleat material	kg/cm^2	f_{yk}	3620
	Cleat flange width	mm	w	125
	Cleat thickness	mm	t	12
	Root radius of cleat	mm	r	14.0
	$X = t\ x \sqrt{\left[\frac{f_{yk}}{1.19\ f_{ck}{}'}\right]}$	cm	$X = \left\{t\ x \sqrt{\left[\frac{f_{yk}}{1.19\ f_{ck}{}'}\right]}\right\} = \left\{(12\ x\ 10^{-1})\ x \sqrt{\left[\frac{3620}{1.19\ x\ 203.9}\right]}\right\}$	4.635
4	Cleat 1 length	cm		17
	Effective width	cm		4.9176
	Force transferred by Cleat 1	kg		20282
	Number of Cleat 1	No.		3
	Cleat 1 capacity	kg		60846
5	Cleat 2 length	cm		17
	Effective width	cm		4.9176
	Force transferred by Cleat 2	kg		20282
	Number of Cleat 2	No.		3
	Cleat 2 capacity	kg		60846
6	Total capacity of both cleats 1 & 2	kg	Total capacity in weight = 60846 + 60846	121692
		kN	$F = mg = \left[\frac{(121692\ x\ 9.81)}{1000}\right] = 1193.80\ kN, hence\ OK$	1193.80
	BOLT DESIGN As per appendix 8 of the Technical Specification, Schedule 4: The design properties of the soil and concrete are:			
1	Compressive force for bolt design (50% load)	kg		48418
2	Tension force for bolt design (50% load)	kg		37064
3	Area of one bolt	cm^2		2.01
	Bolt shear stress	kg/cm^2		3671
	Bolt shear strength	kg	B_1	132811
4	Stub bearing stress	kg/cm^2		7189
	Bolt bearing stress	kg/cm^2		7189
	Stub thickness	mm		18
	Stub bearing strength	kg	B_2	186332
5	Cleat bearing stress	kg/cm^2		7189
	Bolt bearing stress	kg/cm^2		7189
	Cleat thickness	mm		12
	Cleat bearing strength	kg	B_3	248443
6	Bolt strength	kg	Min (B_1, B_2, B_3) = 132811 kg, Hence, OK	132811

FOUNDATION DESIGN CALCULATION FOR AP 104/5

Ultimate Tower Reactions at Base for +3MBE

Load Case	Joint No.	Load Case No.	Nature of Stub force	Inclined forces on foundation (kN)		
				Compressive/ Uplift	Long side thrust	Transversal side thrust
5	41S	C1- TW NC NC V_{max}	Compressive	847.51	0.45	12.475
6	43S	C1- TW NC NC V_{min}	Uplift	569.05	0.30	12.92
7	44S	C2- BWC MCR Br-V_{min}	Compressive	257.69	20.77	31.62
8	41S	C2- BWC BCR Br-V_{min}	Compressive	491.57	26.19	10.50
Load Case	**Joint No.**	**Load Case No.**	**Nature of Stub force**	**Inclined forces on foundation (kg)**		
				Compressive/ Uplift	**Long side thrust**	**Transversal side thrust**
5	41S	C1- TW NC NC V_{max}	Compressive	86421	46	1272
6	43S	C1- TW NC NC V_{min}	Uplift	58027	30	1317
7	44S	C2- BWC MCR Br-V_{min}	Compressive	26277	2118	3224
8	41S	C2- BWC BCR Br-V_{min}	Compressive	50126	2671	1071
Notes: Above foundation forces are ultimate.						

Ultimate Orthogonal Tower Reactions at Base for +3MBE

Load Case	Joint No.	Load Case No.	Nature of Stub force	Inclined forces on foundation (kN)		
				Compressive/ Uplift	Long side thrust	Transversal side thrust
1	41S	C1- TW NC NC V_{max}	Compressive	828.52	109.46	141.12
2	43S	C1- TW NC NC V_{min}	Uplift	555.69	73.41	99.19
3	44S	C2- BWC MCR Br-V_{min}	Compressive	828.52	109.46	141.12
4	41S	C2- BWC BCR Br-V_{min}	Compressive	820.01	108.75	143.63
Load Case	**Joint No.**	**Load Case No.**	**Nature of Stub force**	**Inclined forces on foundation (kg)**		
				Compressive/ Uplift	**Long side thrust**	**Transversal side thrust**
1	41S	C1- TW NC NC V_{max}	Compressive	84485	11162	14390
2	43S	C1- TW NC NC V_{min}	Uplift	56664	7486	10115
3	44S	C2- BWC MCR Br-V_{min}	Compressive	84485	11162	14390
4	41S	C2- BWC BCR Br-V_{min}	Compressive	83617	11089	14646
Notes: Above foundation forces are ultimate.						

Client: Samuel Acidri/KPTL	**Foundation Design Calculation for AP 104/5**				
S/No.	**Ultimate Loadings**	**Orthogonal**	**Inclined**	**Unit**	**Other Details**
1	Compression	84486	86421	kg	Slope = 11.50
2	Tension	56665	58027	kg	
3	Stress Transversal	14646	3224	kg	
4	Stress Longitudinal	11162	2671	kg	

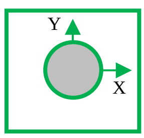
Data for Pile			
S/No.	**Item**	**Description**	**Unit**
1	Number of piles in X-direction	1	No.
2	Number of piles in the Y-direction	1	No.
3	Total number of piles	1	No.
	Type of pile	Simple	
	Type of concrete casting	Cast-in-situ	
	Diameter of pile	0.900	m
	Total length of pile	13.80	m
	Length above scour level	0.00	m
	Spacing between pile	0.00	m
	Volume of pile/leg	8.78	m^3
	Volume of bulb/leg	0.00	m^3

Chimney Details					
S/No.	**Item**	**Description**	**Unit**		
1	Muff height	0.50	m		
2	Chimney height below ground Level	0.00	m		
	Description	**Depth**	**Transverse**	**Longitudinal**	**Unit**
3	Width of the chimney		1.20	1.20	m
a)	Step 1	0.00	1.20	1.20	m
b)	Step 2	0.00	1.20	1.20	m
c)	Step 3	0.00	1.20	1.20	m

Pile Cap Details				
S/No.	**Description**		**Details**	**Unit**
1	Depth of pile cap		0.70	m
2	Side extension		0.15	m
3	Size of pile cap	L	1.20	m
		B	1.20	m
4	Lean pad		0.05	m

Self-Weight of Concrete						
S/No.	**Description**	**Vol/Leg**	**Concrete Dry Density**	**compression weight**	**Concrete wet Density**	**Uplift**
		(m^3)	**(kg/m^3)**	**(kg)**	**(kg/m^3)**	**(kg)**
1	Muff volume	0.72	2300	1656	1300	936
2	Volume of chimney	0.00	2300	0.0	1300	0
3	Volume of Step 1	0.00	2300	0.0	1300	0
4	Volume of Step 2	0.00	2300	0.0	1300	0
5	Volume of Step 3	0.00	2300	0.0	1300	0
6	Lean concrete	0.04	2300	92	1300	52
7	Volume of pile cap/leg	1.01	2300	2318	1300	1310
	Total	**1.77**		**4067**		**2299**

Weight of Superimposed soil (Depth of pile cap below GL 0.00)						
S/No.	**Description**	**Vol/Leg**	**Dry Density**	**Dry Weight**	**Wet Density**	**Wet Weight**
		(m^3)	**(kg/m^3)**	**(kg)**	**(kg/m^3)**	**(kg)**
1	Weight of soil	0.00	1835	0.0	835	0.0
2	Dead weight factor soil and concrete (for compression load only). For super imposed loads only and not on piles				1.33	-

Load at Pile Cap Bottom				
1	Total vertical load	89894	Stub CLS =0.150	
2	Total uplift load	54366		
3	M_y	3546		
4	M_x	2938		

Geometry of the Pile						
Geometry of the Pile					**Distribution of Loads per Pile**	
Pile No.	**X**	**Y**	**X^2**	**Y^2**	**Compression**	**Uplift**
1	0.000	0.000	0.000	0.000	89894	-54366
2	0.000	0.000	0.000	0.000	89894	-54366
3	0.000	0.000	0.000	0.000	89894	-54366

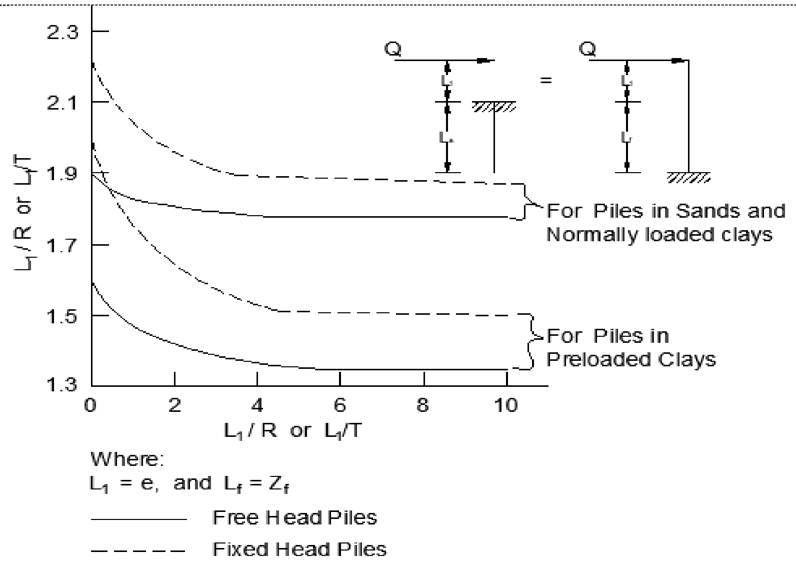
Geometry of the Pile						
Geometry of the Pile					**Distribution of Loads per Pile**	
Pile No.	**X**	**Y**	**X^2**	**Y^2**	**Compression**	**Uplift**
4	0.000	0.000	0.000	0.000	89894	-54366
5	0.000	0.000	0.000	0.000	89894	-54366
6	0.000	0.000	0.000	0.000	89894	-54366
7	0.000	0.000	0.000	0.000	89894	-54366
8	0.000	0.000	0.000	0.000	89894	-54366
9	0.000	0.000	0.000	0.000	89894	-54366
10	0.000	0.000	0.000	0.000	89894	-54366
11	0.000	0.000	0.000	0.000	89894	-54366
12	0.000	0.000	0.000	0.000	89894	-54366
13	0.000	0.000	0.000	0.000	89894	-54366
14	0.000	0.000	0.000	0.000	89894	-54366
15	0.000	0.000	0.000	0.000	89894	-54366
16	0.000	0.000	0.000	0.000	89894	-54366
			0.00	**0.00**	**89894**	**54366**
				OLF =	1	
				Safe Load per Pile =	89894	54366

Description		**Detail**	**Description**
For Sandy soil			**For Clayey soil**
End bearing $Q_u = Q_f + Q_b$			
$Q_b =$	$A_p \; x \left(\frac{1}{2} \; x \; D \; x \; r \; x \; N_r\right) +$ $A_p \; x \left(r \; x \; L \; x \; N_q\right) +$ $A_a \; x \left(\frac{1}{2} \; x \; D_u \; x \; n \; x \; r \; x \; N_r\right) +$ $A_a \; x \left(r \; x \; N_q \; x \; (L_1 + L_2 \ldots L_n)\right) +$		$A_p \; x \; N_c + C_p +$ $A_a \; x \; N_c + C'_a +$ $C_a' \; x \; A_s' +$ $Alpha \; x \; C_a \; x \; A_s$
$Q_f =$	$K \; x \; P_{di} \; x \; \tan\delta \; x \; A_{si}$ $(Due\ to\ friction)$ or $\frac{1}{2} \; x \; P_i \; x \; D \; x \; r \; x \; K \; x \; \tan\delta \; x \left({L_1}^2 + L^2 - {L_n}^2\right)$ (In case of under reamed pile)		
$A_p =$	Area of Pile	0.64 m^2	C_p = Cohesion along the pile
$D =$	Diameter of Pile	0.90 m^2	C'_a = cohesion at the Bulb
$L =$	Length of Pile	15.0 m	C_a = Average cohesion
$D_u =$	Diameter of under reamed	0.0 m	
$A_a =$	Area of Bulb	0.0 m^2	
$\emptyset =$	Angle of internal friction		
$\delta =$	(Delta) Angle of wall friction		
$L_n =$	Depth of last under-ream		
$r =$	Density of soil		
$L_1 =$	Depth of centre first under-ream		

Calculation of friction resistance for soil data received from site													
				For Sandy Soil									
Layer	**Differential Depth**	**Pile Diameter**	**r**	**ϕ**	$\boldsymbol{\delta}$	$\frac{L}{D} =$	**Pressure at base of pile**	**Effective Depth**	$P_d \ x \ Depth$	**K**	**E**	**As**	$F_s \ x \ A_{si}$
	m	m	kg/m^3	°	(= ∅)	20.00	kg/m^2				kg/m^2	m^2	
Pile length above GL	1.20	0.90	0	0	0	1.33	0	1.20	0	0.00	0	3.39	0
2	2.00	0.90	835	0	0	3.56	1670	2.00	1670	0.00	0	5.65	0
3	2.00	0.90	835	0	0	5.78	3340	2.00	5010	1.00	0	5.65	0
4	1.00	0.90	886	25.5	25	6.89	4226	1.00	3783	1.00	1764	2.83	4988
5	1.00	0.90	886	27	27	8.00	5112	1.00	4669	1.00	2379	2.83	6726
6	1.00	0.90	886	27	27	9.11	5998	1.00	5555	1.00	2830	2.83	8003
7	1.00	0.90	886	27	27	10.22	6884	1.00	6441	1.00	3282	2.83	9279
8	1.00	0.90	886	27	27	11.33	7770	1.00	7327	1.00	3733	2.83	10556
9	1.00	0.90	886	27	27	12.44	8656	1.00	8213	1.00	4185	2.83	11832
10	1.00	0.90	886	27	27	13.56	9542	1.00	9099	1.00	4636	2.83	13108
11	1.00	0.90	886	27	27	14.67	10428	1.00	9985	1.00	5088	2.83	14385
12	1.80	0.90	886	27	27	16.67	12023	1.80	20206	1.00	10295	5.09	29109
13		0.00			0	0.00	12023	0.00	0		0	0.00	0
	15.00												**Total = 107987**

Calculation of End Bearing			
For End of Pile in Sandy Soil			
S/No.	**Item Description**	**Detail**	**Unit**
1	For last layer i.e. where the pile rests	27	
2	Corrected N_r	10.12	
3	Corrected N_q	30.00	
4	Density of soil for last layer	886	kg/m^3
5	$A_p \; x \; \frac{1}{2} \; x \; D \; x \; r \; x \; N_r =$	2567	
6	$A_p \; x \left(r \; x \; L \; x \; N_q\right) =$	233351	
7	Total end bearing capacity of pile = Q_b	235917	
8	Total friction capacity of pile = Q_f	107987	
9	Submerged weight of single pile = W_s	11413	
10	Dry weight of single pile = W_d	20192	
11	**Check for Uplift**		
a)	Total uplift capacity = Q_f	107987	
b)	Total uplift load to be resisted = $U - W_s$	42953	
c)	Factor of safety (FoS) under uplift load =	2.51	
12	**Check for Compression**		
a)	Total compression capacity = $Q_b + Q_f$	343904	
b)	Total compression load to be resisted = C+W_d	110087	
c)	Factor of safety (FoS) under compression load =	3.12	

Calculation of depth of Fixity			
S/No.	**Item Description**	**Detail**	**Unit**
1	Grade of Concrete	30	N/mm^2
2	Grade of reinforcement steel used	460	N/mm^2
For Sandy Soil			
	$T = \sqrt[5]{\left(\frac{EI}{K_1}\right)}$, where by:		
	E = Modulus of elasticity of concrete		
	I = Moment of inertia of Pile		
	K_1 = From Table 2		
	T =		
	L_1 = length of pile above ground		



	From the figure above;		
	$L_1/T \ =$	0.336	
	$L_f/T \ =$	1.88	
	Length of Fixity, $L_f \ =$	670.965	cm
		6.710	m
	Say =	**6.710**	m
	Check for deflection		
	Transverse side thrust on pile cap	14646	
	Longitudinal side thrust on pile ca	11162	
	Number of piles	1	
	Transverse side thrust on each pile	14646	
	Longitudinal side thrust on each side	11162	
	Resultant stress transverse (ST) on single pile (Q)	18415	
	Deflection of Pile,	0.55	cm
	$Q \ x \ \frac{(L_f)^3}{EI}$	**5.5**	mm
	Deflection Limit	50	mm
		OK	
	Ultimate Loads on single pile for Pile Design		
	Maximum moment on fixed head pile, $m \ x \ Q \ x \ \frac{(L_1 + L_f)}{2}$		
	Reduction factor (m)	0.45	m
	Transverse side thrust on pile (Q_1)	14646	kg
	Transverse moment on fixed head pile	**52133**	kgm
	Longitudinal side thrust on pile (Q_2)	11162	kg
	Longitudinal moment on fixed head pile	**39730**	kgm

Design of Pile

S/No.	Item Description	Detail	Unit		
1	Grade of Concrete used, f_{ck}	30	N/mm^2		
2	Grade of reinforcement steel used, f_{yk}	460	N/mm^2		
	Length of fixity	6710	mm		
	Length above ground	1200	mm		
	Diameter of Pile, D	900	mm		
	$\frac{L_{eff}}{D}$	8.79			
	Length of Pile	Short pile			
	Area of Pile	636173	mm^2		
	Pile Type	Free-Head			Tomlinson

S/No.	Description	Detail	Unit	Uplift	Compression
1	Length of pile up to fixity from pile top	L_{eff}	mm	7910	
2	Load			NC	
3	Compression		kg	89894	
4	Tension		kg	54366	
				TR NC	LG NC
5	Side thickness		kg	14646	11162
6	Moment due to side thickness		kgm	52133	39730
7	Moment due to water current		kgm	0	-
8	Total moment		kgm	52133	39730
S/No.	**Description**	**Detail**	**Unit**	**Uplift with Bending**	**Compression with Bending**
1	Grade of Concrete used, f_{ck}	f_{ck}	N/mm^2	30	30
2	Grade of reinforcement steel used, f_{yk}	f_{yk}	N/mm^2	460	460
3	Concrete cover	C_c	mm	50	50
4	Uplift/compressive load	P_u	N	533330	881864
5	Transverse moment		Nmm	511425492	511425492
6	Longitudinal moment		Nmm	389755839	389755839
7	Pile area	A_g	mm^2	636173	636173
8	Diameter of bars	dia	mm	25	25
9	Number of bars		No.	17	17
10	Steel area	A_s	mm^2	8345	8345
11	Area of concrete, $A_c = A_g - A_s$	A_c	mm^2	627828	627828
12	$P_t = \frac{100\ x\ A_s}{CHW\ Area}$			1.312	1.312
13	Check for 0.4% minimum steel			OK	

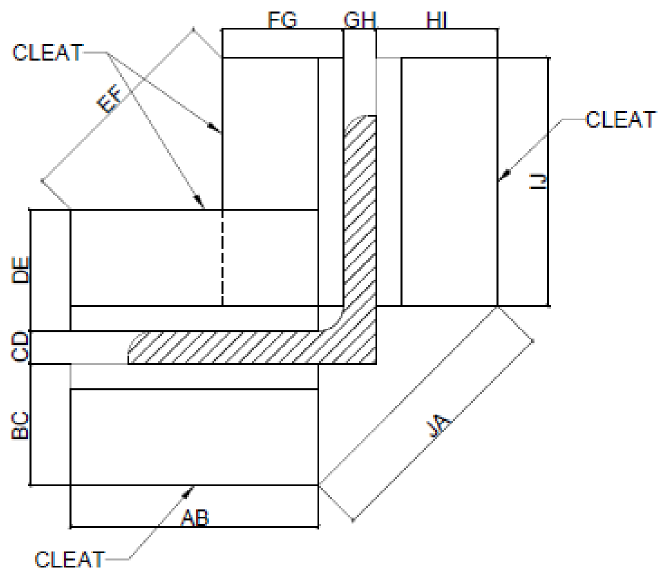
Biaxial bending capacity of Pile (Refer to PROKON Design & analysis)				
S/No.	**Description**	**Unit**	**Required rebar**	**Provided rebar**
1	CASE1: Compression with Bending	mm^2	2893	8345
	Factor of Safety: $FoS = \frac{Provided\ Rebar}{Required\ Rebar} = \left(\frac{8345}{2893}\right)$	FoS	2.88	-
2	CASE2: Uplift with Bending	mm^2	6670	8345
	Factor of Safety: $FoS = \frac{Provided\ Rebar}{Required\ Rebar} = \left(\frac{8345}{6670}\right)$	FoS	1.25	-

Design of Pile Cap

$f_{yk} = 460\ MPa$ $f_{ck} = 30\ MPa$	$\frac{d'}{D} = 0.114$	$b = 1200\ mm$ $d = 1200\ mm$

Orthogonal Forces on Pile Cap (kN)					
S/No.	**Description**	**Case 1**	**Case 2**	**Case 3**	**Case 4**
1	Compression (+) / Tension (-)	828.52	-555.69	828.52	820.01
	Longitudinal	109.46	73.41	109.46	108.75
	Transverse	141.12	99.19	141.12	143.63
	Lever Arm	2.225	2.225	2.225	2.225
	Moment Longitudinal (kNm)	243.5485	163.33725	243.5485	241.96875
	Moment Transverse (kNm)	313.992	220.69775	313.992	319.57675
	Moment Uniaxial (kNm)	557.5405	384.035	557.5405	561.5455
	Moment Uniaxial (Nmm)	557540500	384035000	557540500	561545500
	$\frac{P_u}{(f_{ck}\ x\ b\ x\ d)}$	0.01918	0.01286	0.01918	0.01898
	Minimum steel (%)	0.4	0.4	0.4	0.4
	Diameter of bars	25 / 10	25 / 10	25 / 10	25 / 10
	Number of bars	17 / 16	17 / 16	17 / 16	17 / 16
	Area of steel provided	9601	9601	9601	9601
	$P_t = \frac{100\ x\ A_{st}}{(b\ x\ d)}$	**0.67**	**0.67**	**0.67**	**0.67**
	Check for minimum steel	**OK**	**OK**	**OK**	**OK**
	P_t/f_{ck}	0.0223	0.0223	0.0223	
	$From\ Chart; \frac{M_u}{f_{ck}\ x\ b\ x\ d^2}$	0.039	0.028	0.039	0.039
	Hence, M_u capacity (Nmm)	2021760000	1451520000	2021760000	2021760000
	Factor of Safety (FoS) $= \frac{M_u\ \text{capacity (Nmm)}}{\text{Moment Uniaxial (Nmm)}}$	**3.626**	**3.780**	**3.626**	**3.60**

STUB DESIGN FOR AAP 104/5				
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
1	**Tower Loads (Ultimate) for maximum body extension**			
a)	Compressive weight	kg		86421
	Compressive Load	kN	$F = mg = \left[\frac{(86421\ x\ 9.81)}{1000}\right]$	847.79
	Side thrust (transversal)	kN		31.62
	Side thrust (Longitudinal)	kN		20.77
b)	Tensile weight	kg		58007
	Tensile Load	kN	$F = mg = \left[\frac{(58007\ x\ 9.81)}{1000}\right]$	568.05
	Side thrust (transversal)	kN		21.62
	Side thrust (Longitudinal)	kN		20.77
2	**Stub Details**			
	Stub arrangement	No.		1 single angle
	Stub material			High Tensile (HT)
	Stub section	mm		125 x 125 x 12
	Stub depth inside concrete	mm		2243
3	**Cleat Details**			
	Cleat arrangement	No.		2 (back to back)
	Cleat material			HT
	Cleat section	mm		90 x 90 x 7
	Cleat 1 length	mm	L_1	125
	Cleat 1 number	No.		3
	Cleat 2 length	mm	L_2	125
	Cleat 2 number	No.		3
4	**Bolt Details**			
	Diameter of bolts	mm		16
	No. of bolts	No.		3
5	**Concrete Details**			
	Grade of concrete, f_{ck}	N/mm^2	f_{ck}	25
6	**Stub Area Check as per ASCE 52**			
	Stub flange width	mm		125
	Stub thickness	mm		12
	Stub material, f_{yk}	kg/cm^2		3620
	Stub area provided	cm^2		28.912
	Compressive weight	kg	P	86421
	Transversal side thickness weight	kg		3223
	Longitudinal side thickness weight	kg		2117
	Resistance side thickness weight	kg	V	3856
	Stub area required	cm^2		25.29, OK
	Tensile weight	kg	P	58027
	Transversal side thickness weight	kg		3223
	Longitudinal side thickness weight	kg		2117
	Resistance side thickness weight	kg	V	3856
	Stub area required	cm^2		17.45, OK
7	**Check for stub length in compression**			



	Depth of stub in concrete	cm	$D_{stub\ (concrete)}$	214.8
	Punching shear stress for compression	N/mm²	$\left[0.35\ x\ \sqrt{f_{ck}}\right] = \left[0.35\ x\ \sqrt{25}\right]$	1.75
		kg/cm²	$V_{shear\ (comp)} = \left[0.35\ x\ \sqrt{f_{ck}}\right] = \left[0.35\ x\ \sqrt{\left(\frac{25}{9.81\ x(\ 0.1)^2}\right)}\right]$	17.838
	Perimeter for shear	cm	$P = AB + BC + CD + DE + EF + FG + GH + HI + IJ + JA$ $P = 125 + 90 + 12 + 90 + 70 + 90 + 12 + 90 + 125 + 165$	86.9
	Therefore, punching shear strength	kg	$V_{shear\ (strength)} = \left(D_{stub\ (conc)}\ x\ P\ x\ V_{shear\ (comp)}\right) > Compressive\ wt$ $Compressive\ weight = 96836\ kg$ $\Rightarrow V_{shear\ (strength)} = 214.8\ x\ 86.9\ x\ 17.838$ $\Rightarrow V_{shear\ (strength)} = 332967\ kg > 86421\ kg$	332967
	Factor of Safety, FoS		$FoS = \frac{332967\ kg}{86421\ kg} = 3.86 > 1, Hence, SAFE$	3.86
8	**Check for Stub Length in Uplift**			
	Punching shear stress for uplift	kg/cm²	$V_{shear\ (uplift)} = \left[0.28\ x\ \sqrt{f_{ck}}\right] = \left[0.28\ x\ \sqrt{25}\right]$	14.271
	Therefore, punching shear strength	kg	$V_{shear\ (strength)} = \left(D_{stub\ (conc)}\ x\ P\ x\ V_{shear\ (uplift)}\right) > Tensile\ wt$ $Tensile\ weight = 58027\ kg$ $\Rightarrow V_{shear\ (strength)} = 214.8\ x\ 86.9\ x\ 14.271$ $\Rightarrow V_{shear\ (strength)} = 266385\ kg > 58027\ kg$	266385
	Factor of Safety, FoS		$FoS = \frac{266385\ kg}{58027\ \ kg} = 4.60 > 1, Hence, SAFE$	4.60

S/No.	Description	Unit	Calculations/Equations	Results/Remarks
	CLEAT DESIGN AS PER ASCE 52 As per appendix 8 of the Technical Specification, Schedule 4: The design properties of the soil and concrete are:			
1	Ultimate Compression for Cleat design (50% Load)			
	Ultimate compression weight	kg	Weight	43211
	Ultimate compression load	kN	$F = mg = \left[\frac{(43211\ x\ 9.81)}{1000}\right]$	423.9
2	Concrete grade	N/mm²	f_{ck}	25
		N/mm²	$f_{ck}{}' = (0.8\ x\ f_{ck}) = 0.8\ x\ 25$	20
		kg/cm²	$f_{ck}{}' = (0.8\ x\ f_{ck}) = \left[0.8\ x\ \frac{25}{9.81\ x\ (0.1)^2}\right]$	203.9
3	Cleat material	kg/cm²	f_{yk}	3620
	Cleat flange width	mm	w	90
	Cleat thickness	mm	t	7
	Root radius of cleat	mm	r	10.0
	$X = t\ x\ \sqrt{\left[\frac{f_{yk}}{1.19\ f_{ck}{}'}\right]}$	cm	$X = \left\{t\ x\ \sqrt{\left[\frac{f_{yk}}{1.19\ f_{ck}{}'}\right]}\right\} = \left\{(7\ x\ 10^{-1})\ x\ \sqrt{\left[\frac{3620}{1.19\ x\ 203.9}\right]}\right\}$	2.704
4	Cleat 1 length	cm		12.5
	Effective width	cm		3.0519
	Force transferred by Cleat 1	kg		9255
	Number of Cleat 1	No.		3
	Cleat 1 capacity	kg		27766
5	Cleat 2 length	cm		12.5
	Effective width	cm		3.0519
	Force transferred by Cleat 2	kg		9255
	Number of Cleat 2	No.		3
	Cleat 2 capacity	kg		27766
6	Total capacity of both cleats 1 & 2	kg	Total capacity in weight = 27766 + 27766	55532
		kN	$F = mg = \left[\frac{(55532\ x\ 9.81)}{1000}\right] = 1193.80\ kN, hence\ OK$	544.77

S/No.	Description	Unit	Calculations/Equations	Results/Remarks
	BOLT DESIGN As per appendix 8 of the Technical Specification, Schedule 4: The design properties of the soil and concrete are:			
1	Compressive force for bolt design (50% load)	kg		43211
2	Tension force for bolt design (50% load)	kg		29004
3	Area of one bolt	cm^2		2.01
	Bolt shear stress	kg/cm^2		3671
	Bolt shear strength	kg	B_1	88541
4	Stub bearing stress	kg/cm^2		7189
	Bolt bearing stress	kg/cm^2		7189
	Stub thickness	mm		12
	Stub bearing strength	kg	B_2	82814
5	Cleat bearing stress	kg/cm^2		7189
	Bolt bearing stress	kg/cm^2		7189
	Cleat thickness	mm		7
	Cleat bearing strength	kg	B_3	96617
6	Bolt strength	kg	Min (B_1, B_2, B_3) = 82814 kg, Hence, OK	82814

Circular column design

C12

Circular column design by *PROKON.* (CirCol Ver W3.0.06 - 12 May 2016)

Design code : BS8110 - 1997

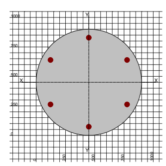
Input tables

General design parameters and loads:

Load Case	Description	Ultimate Limit State Design Loads				
		P (kN)	Mx top (kNm)	My top (kNm)	Mx bot (kNm)	My bot (kNm)
1	COMP	828.52			511.25	389.62
2	UPLIFT	-555.69			511.25	389.62

Ø (mm)	900
d' (mm)	72.5
Lo (m)	7.91
fcu (MPa)	30
fy (MPa)	460

General design parameters:



Given:

d = 900 mm
d' = 72 mm
Lo = 7.910 m
fcu = 30 MPa
fy = 460 MPa

Therefore:

$$A_c = \frac{\pi \cdot d^2}{4} = \frac{\pi \times 900^2}{4} = 636.2\times10^3 \text{ mm}^2$$

$$d_{iax'} = d_{ia} - d' = 900 - 72.5 = 827.500 \text{ mm}$$

$$d_{iay'} = d_{ia} - d' = 900 - 72.5 = 827.500 \text{ mm}$$

Assumptions:
(1) The general conditions of clause 3.8.1 are applicable.
(2) The section is symmetrically reinforced.
(3) The specified design axial loads include the self-weight of the column.
(4) The design axial loads are taken constant over the height of the column.

Design approach:

The column is designed using the following procedure:
(1) The column design charts are constructed.
(2) The design axis and design ultimate moment is determined .
(3) The steel required for the design axial force and moment is read from the relevant design chart.
(4) The area steel perpendicular to the design axis is read from the relevant design chart.
(5) The procedure is repeated for each load case.
(6) The critical load case is identified as the case yielding the largest steel area about the design axis.

Through inspection:
Load case 2 (UPLIFT) is critical.

Check column slenderness:

End fixity and bracing for bending about the X-X axis:
The column is braced.
$\therefore$ ßx = 0.80 *Table 3.21*

End fixity and bracing for bending about the Y-Y axis:
The column is braced.
$\therefore$ ßy = 0.80 *Table 3.21*

Effective column height:

$$l_{ex} = ß_x \cdot L_o$$
$$= .8 \times 7.91$$
$$= 6.328 \text{ m}$$

$$l_{ey} = ß_y \cdot L_o$$
$$= .8 \times 7.91$$
$$= 6.328 \text{ m}$$

Column slenderness about both axes:

$$\lambda_x = \frac{l_{ex}}{d_{ia}}$$
$$= \frac{6.328}{.9}$$
$$= 7.031$$

$$\lambda_y = \frac{l_{ey}}{d_{ia}}$$

$$= \frac{6.328}{.9}$$

$$= 7.031$$

Minimum Moments for Design:

Check for mininum eccentricity: 3.8.2.4
For bi-axial bending, it is only necessary to ensure that the eccentricity exceeds the minimum about one axis at a time.

For the worst effect, apply the minimum eccentricity about the minor axis:
Use emin = 20mm

$$M_{min} = e_{min} \cdot N$$

$$= .02 \times - 555.69$$

$$= -11.1138 \text{ kNm}$$

Check if the column is slender: 3.8.1.3

$\lambda x = 7.0 < 15$

$\lambda y = 7.0 < 15$

∴ The column is short.

Initial moments:

The initial end moments about the X-X axis:
M1 = Smaller initial end moment = 0.0 kNm
M2 = Larger initial end moment = 511.2 kNm

The initial moment near mid-height of the column : 3.8.3.2

$$M_i = -0.4 \cdot M_1 + 0.6 \cdot M_2$$

$$= -0.4 \times 0 + 0.6 \times 511.25$$

$$= 306.750 \text{ kNm}$$

$$M_{i2} = 0.4 \cdot M_2$$

$$= 0.4 \times 511.25$$

$$= 204.500 \text{ kNm}$$

∴ Mi ≥ 0.4M2 = 511.2 kNm

The initial end moments about the Y-Y axis:
M1 = Smaller initial end moment = 0.0 kNm
M2 = Larger initial end moment = 389.6 kNm

The initial moment near mid-height of the column : 3.8.3.2

$$M_i = -0.4 \cdot M_1 + 0.6 \cdot M_2$$
$$= -0.4 \times 0 + 0.6 \times 389.62$$
$$= 233.772 \text{ kNm}$$

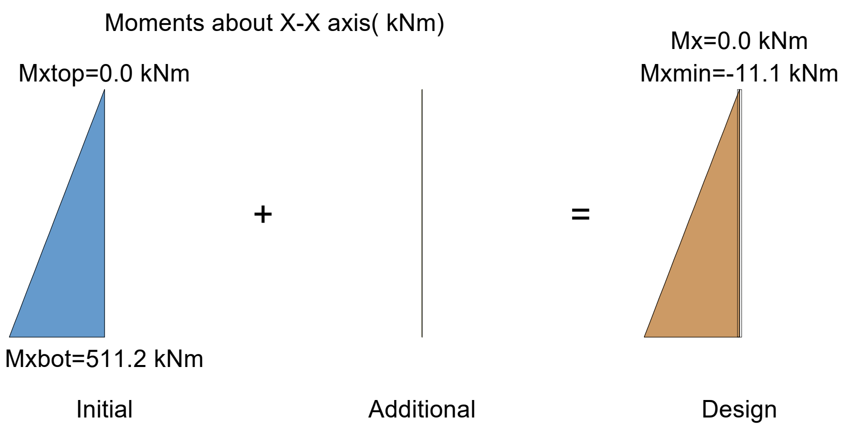
$$M_{i2} = 0.4 \cdot M_2$$
$$= 0.4 \times 389.62$$
$$= 155.848 \text{ kNm}$$

∴ Mi ≥ 0.4M2 = 389.6 kNm

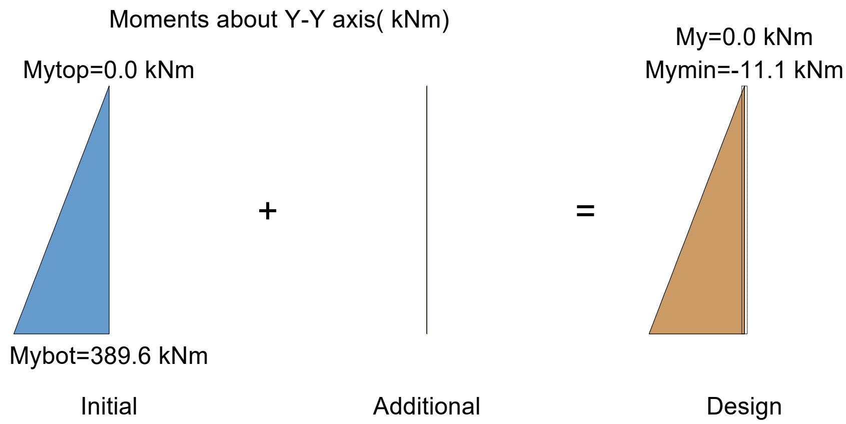
Design ultimate load and moment:

Design axial load:
P_u = -555.7 kN

Moment distribution along the height of the column for bending about the X-X:
At the top, Mx = 0.0 kNm
Near mid-height, Mx = 0.0 kNm
At the bottom, Mx = 0.0 kNm



Moment distribution along the height of the column for bending about the Y-Y:
At the top, My = 0.0 kNm
Near mid-height, My = 0.0 kNm
At the bottom, My = 0.0 kNm



Design of column section for ULS:

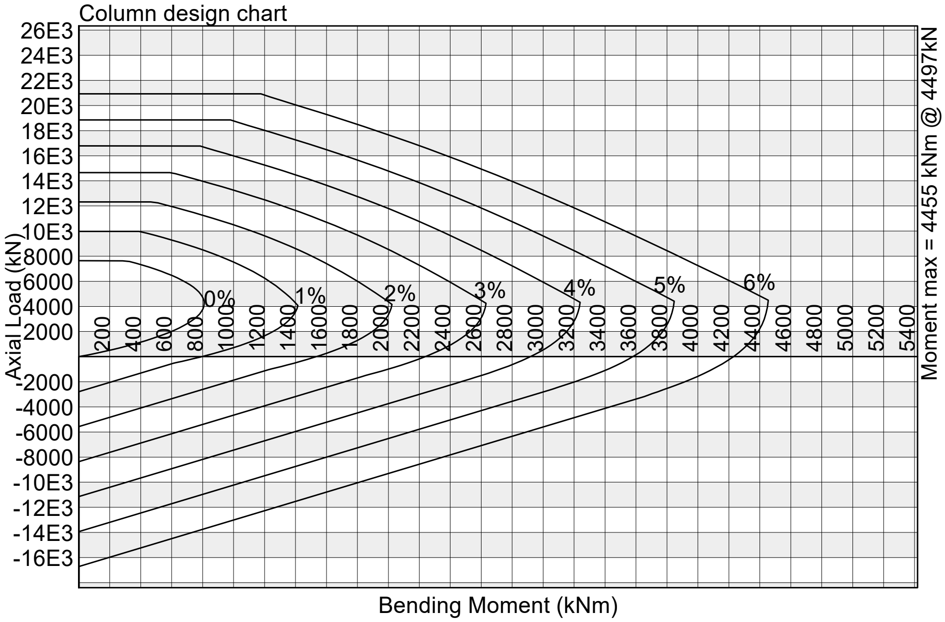
Through inspection:
The critical section lies at the bottom end of the column.

The column is bi-axially bent: the moments are therefore added vectorially to obtain the final design moment:

$$
\begin{aligned}
M' &= \sqrt{{M_x}^2 + {M_y}^2} \\
&= \sqrt{511.25^2 + 389.62^2} \\
&= 642.791
\end{aligned}
$$

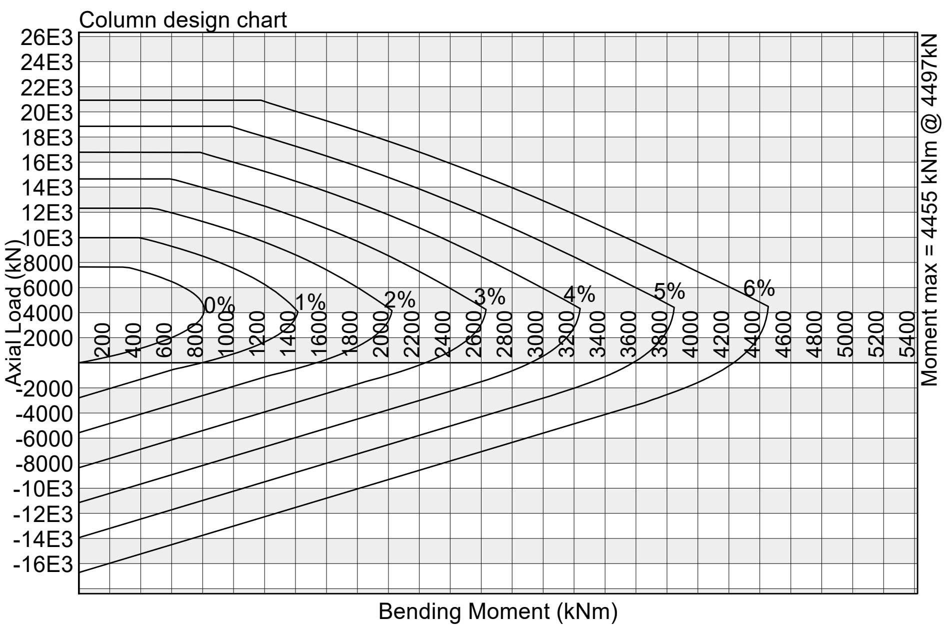
Design axial load:
P_u = -555.7

For bending about the design axis:



From the design chart, Asc = 6670 = 1.05%

Design chart for bending about any axis:



Summary of design calculations:

Design results for all load cases:

Load case	Axis	N (kN)	M1 (kNm)	M2 (kNm)	Mi (kNm)	Madd (kNm)	Design	M (kNm)	M' (kNm)	Asc (mm²)
1	X-X Y-Y	828.5	0.0 0.0	511.2 389.6	511.2 389.6	0.0 0.0	X-X Bottom	511.2 389.6	642.8	2893 (0.45%)
2	X-X Y-Y	-555.7	0.0 0.0	511.2 389.6	511.2 389.6	0.0 0.0	X-X Bottom	511.2 389.6	642.8	6670 (1.05%)

Load case 2 (UPLIFT) is critical.

FOUNDATION DESIGN CALCULATION FOR AP 108/20

Ultimate Tower Reactions at Base for +12MBE

Load Case	Joint No.	Load Case No.	Nature of Stub force	Inclined forces on foundation (kN)		
				Compressive/ Uplift	Long side thrust	Transversal side thrust
17	41S	C1- TW NC NC V_{max}	Compressive	886.67	0.70	19.011
18	43S	C1- TW NC NC V_{min}	Uplift	594.45	0.06	19.53
19	44S	C2- BWC MCR Br-V_{min}	Compressive	280.26	16.55	31.51
20	42S	C2- BWC BCL Br-V_{min}	Uplift	133.57	21.56	23.14
Load Case	**Joint No.**	**Load Case No.**	**Nature of Stub force**	**Inclined forces on foundation (kg)**		
				Compressive/ Uplift	**Long side thrust**	**Transversal side thrust**
17	41S	C1- TW NC NC V_{max}	Compressive	90414	72	1939
18	43S	C1- TW NC NC V_{min}	Uplift	60617	6	1991
19	44S	C2- BWC MCR Br-V_{min}	Compressive	28579	1688	3213
20	42S	C2- BWC BCL Br-V_{min}	Uplift	13621	2198	2360
Notes: Above foundation forces are ultimate.						

SOIL PROPERTIES FOR FOUNDATION DESIGN

Please refer schedule4: Design-properties of soil and concrete of technical specification.

S/No.	Soil Type	Poor Soil
1	Assumed mass of Earth for foundations, γ_s (kg/m^3)	1600
2	Assumed mass of rock for foundations, γ_r (kg/m^3)	-
3	Assumed mass of concrete for foundations, γ_c (kg/m^3)	2300
4	Assumed ultimate bearing capacity for foundations under specified maximum ultimate loading, including factor of safety:	
(a)	t/m^2	30
(b)	kN/m^2	294.19
5	Ultimate shear stress in rock, τ	
(a)	t/m^2	-
(b)	kN/m^2	-
6	Assumed angle to vertical of frustum of earth resisting uplift (angle of Repose).	30°
7	Assumed angle to vertical of frustum of earth resisting uplift (angle of repose) considered in foundation design, Ø.	25°

Client: Samuel Acidri/KPTL			**FOUNDATION DESIGN CALCULATION FOR AP 108/20**	
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
1	Min. back-to-back distance (TR)	m		11.015
	Min. back-to-back distance (LG)	m		9.868
2	Working point			At top of footing
3	Leg-angle (Transverse), Ø	Degrees	θ_T	8.825
	Leg-angle (Longitudinal), Ø	Degrees	θ_L	7.496
	True Angle	Degrees	θ	8.148
	Length factor		$1/\cos^2\theta$	1.02
	Chimney/Column			Inclined
4	**Maximum Loads as per FAT Test**			
a)	P-compression, P_{comp}	kN	P_{comp}	886.67
b)	P-tension, $P_{tension}$	kN	$P_{tension}$	594.45
c)	Shear -transversal, Tr	kN	Tr	33.27
c)	Shear -longitudinal, Lg	kN	Lg	27.88
5	Footing type			Double slab
6	**Footing Dimensions**			
a)	Footing depth, H	m	H	3.5
b)	Footing dimensions, L x W x H	m	$L\ x\ W\ x\ H$	2.64 x 2.64 x 0.35
c)	Step, L x W x H	m	$L\ x\ W\ x\ H$	1.2 x 1.2 x 0.25
d)	Footing base, B	m	B	2.64
e)	Lean pad height, P	m	P	0.05
7	**Column/Chimney Dimensions**			
	Size, W	m	W	0.55 x 0.55
	Muff, M	mm	M	800
	Chimney height, H_1	mm	H_1	2850
	Base thickness, D	mm	D	350
	Step depth, D_1	mm	D_1	250
	Step width, B_1	mm	B_1	1200
	Working point height, Wp	mm	W_p	600
8	**Footing Reinforcement**			
	Bottom	No. -Dia		15 No, Dia 14 @ 180 c/c/
	Top -step	No. -Dia		6 No, Dia 14 @ 220 c/c
	Top -base	No. -Dia		10 No, Dia 14 @ 280 c/c
9	**Column/Chimney reinforcement**			
	Main internal rebars	No. -Dia		8 No, Dia 16
	Main corner rebars	No. -Dia		4 No, Dia 20
	Links	Dia- Spc		Dia 8 @ 225 c/c
10	**Material Data**			
	Concrete density in RCC and PCC (dry), γ_c	kg/m^3	γ_c	2300
	Concrete cover to bottom surface, c	mm	c	100
	Concrete cover to top and side surfaces, c'	mm	c'	50
	Characteristic concrete strength, f_{ck}	MPa	f_{ck}	C25
	Characteristic steel strength, f_{yk}	MPa	f_{yk}	f_{yk} 500
11	**Data for Checks (Soil cone from edge)**			
	Height of soil cone, H_b	m	H_b	2.8
	Effective soil weight (soil cone + excavation pit weights)	kN	$W_s = soil\ cone\ weight +\ excavation\ pit\ weight$	687.92
	Effective concrete weight, W_c	kN	W_c	30.593
	-in muff		$\gamma_c\ x\ M\ x\ W^2 = \left[\left(\frac{2300x9.81}{1000}\right)\ x\ 0.8\ x\ 0.55^2\right]$	5.460
	-in soil		$\left[(H_1xW^2) + \left((D\ x\ B^2) + (D_1xB_1{}^2)\right)\right]x(\gamma_c - \gamma_s)$ $(H_1xW^2) = (2.85\ x\ 0.55^2) = 0.862125$ $(D\ x\ B^2) = (0.35\ x\ 2.64^2) = 2.43936$ $(D_1xB_1{}^2) = (0.25\ x\ 1.20^2) = 0.36$ $(\gamma_c - \gamma_s) = (22.563 - 15.696) = 6.867$ $\gamma_c = \left(\frac{2300\ x\ 9.81}{1000}\right) = 22.563\ \&\ \gamma_s = \left(\frac{1600\ x\ 9.81}{1000}\right) = 15.696$ $\Rightarrow [0.862125 + (2.43936 + 0.36)]x\ 6.867$	25.143

Client: Samuel Acidri/KPTL			**FOUNDATION DESIGN CALCULATION FOR AP 108/20**	
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
12	**Uplift check**			
	Load case	No.		18
	Load	kN		594.45
	Factor of safety (FoS)			1.209
13	**Overturning check**			
	Load case	No.		18
	Tensile load (Uplift Load)	kN	Uplift	594.45
	Shear force- transversal, (Tr)	kN	Tr	19.53
	Shear force- longitudinal, (Lg)	kN	Lg	0.06
	Moment due to uplift	kNm	$M_{Uplift} = Uplift\ x\ \cos^2\theta\ x\ \frac{B}{3}$ $\Rightarrow M_{Uplift} = \left[594.45\ x\ (\cos 25)^2\ x\ \left(\frac{2.64}{3}\right)\right]$	513.953
	Moment due to side thrust	kNm	$M_{ST} = Max\ (Tr, Lg, ST)\ x\ (M + H - P)$ $\Rightarrow M_{ST} = 19.53\ x\ (0.8 + 3.50 - 0.05)$	83.0025
	Moment due to concrete	kNm	$M_c = \left(W_c\ x \frac{B}{3}\right) = \left(30.593 \text{ x } \frac{2.64}{3}\right)$	26.922
	Resisting moment due to soil	kNm	$M_s = \left[\left(\frac{W_s}{2}\right)\ x\ \left(\frac{5}{6}\right) x\ B\right] = \left[\left(\frac{687.92}{2}\right)\ x\ \left(\frac{5}{6}\right) x\ 2.64\right]$	756.712
	Total overturning moment	kNm	$M_T = M_{Uplift} + M_{ST} - M_c = (513.953 + 83.0025 - 26.922)$ $\Rightarrow Since\ M_T = 570.034 < \ M_s = 756.712, then\ its\ OK.$	570.034
	Factor of safety, FoS		$FoS = \frac{M_s}{M_T} = \left(\frac{756.712}{570.034}\right)$	1.328
14	**Base pressure check**			
	Load case	No.		17
	Compressive load	kN	P_{comp}	886.67
	Shear force (Tr)- transversal force	kN		19.011
	Shear force (Lg)- longitudinal force	kN		0.7
	Balancing height for shear in transversal direction (Tr) -B_{Ht}	m	$B_{Ht\ (Tr)} = \frac{Shear\ force\ (Tr)}{\sqrt{Passive\ pressure}}$ NOTE: Balancing height is greater than chimney/column height. Therefore, it is restricted to the chimney/column height. Passive pressure $(\mathrm{P_p}) = 0.5 \text{ x } \mathrm{k_p} \text{ x } \gamma_s \text{ x W}$ Where: $k_p = (1 + \sin\emptyset)/(1 - \sin\emptyset) = (1 + \sin 25)/(1 - \sin 25) = 0.7663$ $\gamma_s = \left(\frac{1600\ x\ 9.81}{1000}\right) = 15.696\ kN/m^3$ $\mathrm{P_p} = 0.5 \text{ x } \mathrm{k_p} \text{ x } \gamma_s \text{ x W} = (0.5 \text{ x } 0.7663 \text{ x } 15.696 \text{ x } 0.55) = 3.308$ $B_{Ht\ (Tr)} = \frac{Shear\ force\ (Tr)}{\sqrt{Passive\ pressure}} = \frac{19.011}{\sqrt{3.308}}$	1.337
	Moment due to passive transversal force (Tr)- $M_{p\ (Tr)}$	kNm	$\mathrm{M_{passive}}$ (Tr) $M_{p\ (Tr)} = P_p\ x\ (B_{Ht})^2\ x\ \left\{\left(\frac{B_{Ht}}{3}\right) + (H_1 - 0.3 - B_{Ht}) + D + D_1\right\}$	42.937
	Balancing height for shear in longitudinal direction (Lg)	m	$B_{Ht\ (Lg)} = \frac{Shear\ force\ (Lg)}{\sqrt{Passive\ pressure}}$ Passive pressure $(\mathrm{P_p}) = 0.5 \text{ x } \mathrm{k_p} \text{ x } \gamma_s \text{ xW}$ Where: $k_p = (1 + \sin\emptyset)/(1 - \sin\emptyset)$	0.257
	Moment due to passive longitudinal force (Lg)- $M_{p\ (Lg)}$	kNm	$\mathrm{M_{passive}}$ (Lg) $M_{p\ (Lg)} = P_p\ x\ (B_{Ht})^2\ x\ \left\{\left(\frac{B_{Ht}}{3}\right) + (H_1 - 0.3 - B_{Ht}) + D + D_1\right\}$	2.085
	Soil bearing capacity available:	kN/m^2		294.19
	= P/A	kN/m^2	$\frac{P}{A} = \frac{\left(\frac{P_{comp}}{L.F}\ x\ Overload\right)}{B^2}$	129.397
	= P_{ex}/Z	kN/m^2	$\frac{P_{ex}}{Z} = 2\ x\ \left(\frac{P_{comp}}{L.F}\right) x\ \tan\theta\ x\ W_p\ x\ \frac{6}{B^3}$	48.679
	= M/Z (Transversal), $M_{(Tr)}$	kN/m^2	$\{M_{(Tr)} - M_{p\ (Tr)}\}\ x\ \frac{6}{B^3}$ $M_{(Tr)} = Shear\ force\ (Tr) x\ Lever\ arm$ Where: $Lever\ arm = H + M - P$	12.346
	= M/Z (Longitudinal), $M_{(Lg)}$	kN/m^2	$\{M_{(Lg)} - M_{p\ (Lg)}\}\ x\ \frac{6}{B^3}$ $M_{(Lg)} = Shear\ force\ (Lg) x\ Lever\ arm$ Where: $Lever\ arm = H + M - P$	0.290
	= Maximum pressure	kN/m^2	$\frac{P}{A} + \frac{P_{ex}}{Z} + M_{(Tr)} + M_{(Lg)}$	190.712
	= Minimum pressure	kN/m^2	$\frac{P}{A} - \frac{P_{ex}}{Z} - M_{(Tr)} - M_{(Lg)}$	68.082

Client: Samuel Acidri/KPTL			**FOUNDATION DESIGN CALCULATION FOR AP 108/20**	
S/No.	**Description**	**Unit**	**Calculations/Equations**	**Results/Remarks**
	Factor of safety (FoS)		$FoS = \frac{Soil\ bearing\ capacity}{Max. Pressure} = \frac{294.19}{190.712}$	1.543
15	**Design for Bending for Bottom**			
	Load case	No.		17
	Compressive load	kN		886.67
	Shear force- transversal (Tr)	kN		19.011
	Shear force- longitudinal (Lg)	kN		0.7
	Maximum pressure, P_1	kN/m^2	$P_1 = \frac{P}{A} + \frac{P_{ex}}{Z} + Max\left[M_{(Tr)}, M_{(Lg)}\right]$	166.082
	Bending at face of chimney			
	Effective depth		$d_t = \left[(D + D_1) - C_{cover\ (bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$ $\Rightarrow d_t = \left[(350 + 250) - 100 - 14 - \frac{14}{2}\right]$	479
	Bending Moment -design, BM_{Des}	kNm	$BM_{Des} = \left[P_1\ x\ \frac{(B - W)^2}{8}\ x\ B\right]$ $\Rightarrow BM_{Des} = \left[166.082\ x\ \frac{(2.64 - 0.55)^2}{8}\ x\ 2.64\right]$	239.403
	Area of steel required, $A_{st}(Req)$	mm^2	$A_{st}(Req) = \frac{M}{0.87\ f_{yk}\ Z}$ $k = \frac{M}{bd^2\ f_{ck}} = \frac{239.403\ x\ 10^6}{2640\ x\ 479^2\ x\ 25} = 0.016 \leq k_{bal} = 0.167, Hence\ OK.$ $Lever\ Arm = Z = d\left[0.5 + \left(0.25 - \frac{k}{1.134}\right)^{0.5}\right] \ngtr 0.959d$ $\Longrightarrow Z = 479\left[0.5 + \left(0.25 - \frac{0.016}{1.134}\right)^{0.5}\right] = 472.144\ mm$ $0.95d = 0.959\ x\ 479 = 459.361\ mm$ $\Longrightarrow Z = 472.144\ mm \ngtr 0.959d = 459.361\ mm,$ $Hence\ take\ Z = 0.959d = 459.361\ mm, OK.$ $\Longrightarrow A_{st}(Req) = \frac{M}{0.87\ f_{yk}\ Z} = \frac{239.403\ x\ 10^6}{0.87\ x\ 460\ x\ 459.361}$	2269.308
	Area of steel provided, $A_{st}(Prov)$	mm^2	$A_{st}(Prov) = \left(\frac{\pi D^2}{4} x\ No\ of\ bars\right) = \left(\frac{\pi\ x\ 14^2}{4} x\ 15\right)$ $Since\ A_{st\ (Req)} < A_{st\ (prov)}, Hence\ OK.$	2309.071
	Bending Moment -resisting, BM_{Res}	kNm	$BM_{Res} = 0.87 f_{yk}\ x\ A_{st}\ x\ d_t\ \left(1 - \frac{0.9738\ x\ f_{yk}\ x\ A_{st}}{f_{ck}\ x\ B_1\ x\ d_t}\right)$ $\Longrightarrow 0.87 f_{yk}\ x\ A_{st\ (prov)}\ x\ d_t\ \left(1 - \frac{0.9738\ x\ f_{yk}\ x\ A_{st\ (prov)}}{f_{ck}\ x\ B_1\ x\ d_t}\right)$ $= \left[0.87\ x\ 500\ x\ 2309.071 x\ 479\ x\left(1 - \frac{0.9738\ x\ 500\ x\ 2309.071}{25\ x\ 1200\ x\ 479}\right)\right] x 10^{-6}$ $Since, BM_{Res} = 443.487\ kNm > BM_{Des} = 239.403\ kNm, Hence\ OK.$	443.487
	Factor of safety (FoS)		$FoS = \frac{BM_{Res}}{BM_{Des}} = \left(\frac{443.487}{239.403}\right)$	1.853
	Bending at face of step			
	Effective depth	mm	$d_b = \left[(D + D_1) - C_{cover\ (bottom)} - Dia_1 - \frac{Dia_2}{2}\right]$ $\Rightarrow d_b = \left[(350) - 100 - 14 - \frac{14}{2}\right]$	229
	Bending Moment -design, BM_{Des}	kNm	$BM_{Des} = \left[P_1\ x\ \frac{(B - B_1)^2}{8}\ x\ B\right]$ $\Rightarrow BM_{Des} = \left[59.963\ x\ \frac{(4.39 - 1.88)^2}{8}\ x\ 4.39\right]$	207.303
	Area of steel required, $A_{st}(Req)$	mm^2	$A_{st}(Req)$	2172.476
	Area of steel provided, $A_{st}(Prov)$	mm^2	$A_{st}(Prov) = \left(\frac{\pi D^2}{4} x\ No\ of\ bars\right) = \left(\frac{\pi\ x\ 14^2}{4} x\ 29\right)$	4464.203
	Bending Moment -resisting, BM_{Res}	kNm	$\left[0.87 f_{yk}\ x\ A_{st\ (prov)}\ x\ d_b \left(1 - \frac{0.9738\ f_{yk}\ x\ A_{st\ (prov)}}{f_{ck}\ x\ B\ x\ d_b}\right)\right] x 10^{-6}$ $Since, BM_{Res} = 406.241\ kNm > BM_{Des} = 207.303\ kNm, Hence\ OK.$	406.241
	Factor of safety (FoS)		$FoS = \frac{BM_{Res}}{BM_{Des}} = \left(\frac{406.241}{207.303}\right)$	1.96
	Percentage of steel provided in the footing, P_T	%	$P_T = 100\ x\ \frac{A_{st}(Prov)}{Total\ Cross\ sectional\ Area}$ $\Longrightarrow P_T = 100\ x\ \left[\frac{4464.203}{(4390\ x\ 350) + (1880\ x\ 275)}\right]$	0.217