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EVALUATION ON MINIMUM AND MAXIMUM COVER THICKNESS FOR REINFORCED CONCRETE PIPE CULVERT UNDER EMBANKMENT IN JIMMA TOWN

A thesis submitted to the School of Graduate Studies of Jimma University in Partial fulfillment of the requirements for the Degree of Master of Science in Geotechnical Engineering

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November, 2015

Jimma, Ethiopia

Declaration

This thesis is my original work and has not been presented for degree in any other university

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This thesis has been submitted for examination with my approval with university supervisors

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Acknowledgment

First of all I would like to say thanks to Glorious Almighty God for his endless love and gives me health life and makes me to think rationally. Next, my deepest admiration and appreciation goes to Prof. Emer T.Quezon, the advisor of this thesis, for his valuable contribution of idea and continuous evaluation of the thesis with generating rational comment that helps me to finalize the paper. My special thanks also goes to Eng. Efrem Wakjira who motivate and encourage me throughout preparation of this thesis.

Also, I would like to express my deepest gratitude to Ethiopia Road Authority that sponsor me to join this program and be successful in my thesis by providing financial assistance.

I am also great full to Dr Daniel Mekonnen who discharge the fruit full advice and continuous financial and material supply during my study.

Lastly, I would like to appreciate my lovely family for their endless and continuous encouragement in my study, without them this research work would not be finished.

Abstract

In most highway construction of asphalt and gravel road project, requires an installation of pipe culvert, across and along the side of the road under the surface layer.

This research study covered evaluating the minimum and maximum fill embankment height over the top of the pipe and assessment of factors that would affect the strength of pipe culvert like trench width and quality of contact between the pipe and bedding, pipe strength, bedding type, magnitude of lateral pressure, axial thrust. The main objective of the study was to set out the minimum and maximum fill embankment height over the top of the pipe culvert.

The procedure followed was accomplishing laboratory tests to understand the property of backfill materials and apply pipe Pac software for further analysis by using data obtained from laboratory results

Application of different types of loading on pipe culvert has been considered and laboratory tests like compaction test, sieve analysis, hydrometer, specific gravity, and Atterberg limit test, for design and analysis purpose has been carried out. Sample has gathered from the site and enabled to get existing design data.

Stress analysis by using Boussinesq's theory to get the depth at which stress influence has minimum. From the compaction test results of each site, backfill materials from Seka borrow pit has highest percentage relative compaction but Jiren borrow pit has lowest percentage relative compaction. The minimum and maximum cover thickness decided after evaluation was 1.6m and 0.3m respectively.

This research paper concludes the maximum backfill depth for each site were calculated and the values obtained were specific to the site and within the standard specification. It also recommends blending backfill materials that are poor graded, and settlement analysis of foundation under pipe culvert.

Keywords: pipe culvert and backfill height

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ACRONYMS

AASHTO American Association of State Highway and Transportation Officials

ASTM American Society for Testing and Materials

CSA Canadian Standards Association

CHBDC Canadian Highway Bridge Design Code

ERA Ethiopia Road Authority

ETB Ethiopian birr

F.D.D Field dry density

JL1 Jiren quarry site at location 1

JL2 Jiren quarry site at location

LRFD Load and resistance factored design

ML1 Merewa quarry site at location 1

ML2 Merewa quarry site at location 2

OCPA Ontario Concrete pipe Association

SCDOT South Carolina, Department of Transportation

SIDD Standard Installations Direct Design

SL1 Seka quarry site at location 1

SL2 Seka quarry site at location 2

SPIDA Soil-Pipe Interaction Design and Analysis

3EB Three Edge Bearing

γ_b Bulk field density

γ_d Dry field density

CHAPTER ONE

INTRODUCTION

1.1 Background

The design and construction of pipe culverts are among the most important areas of public works in engineering, and like all other engineering projects, they involve various stages of development.

In the past decades, there were different researchers who had evaluated number of issues that had been incorporated in the design and analysis of buried pipe culverts under embankment in the highway road construction. For the purpose of analysis they had categorized the pipes in to rigid and flexible pipes and each of them were specifically having their own subdivisions depending on the material they were made, method of construction, installation conditions, as well as the general behavior they do have after installation due to the different types of loading applications [19].

Rigid pipes are generally considered as pipes that cannot deflect 2% of their diameter before failing. Common rigid pipes include reinforced concrete, non-reinforced concrete and clay as well as other specialized pipe materials. Because rigid pipe do not deflect significantly when loaded, the pipe must be capable of supporting the backfill materials and any additional loads that are applied to it [20].

A number of factors including the minimum and maximum fill embankment height over the top of the pipe, trench width and type of bedding materials, pipe strength, and magnitude of lateral pressure effect. The type of back fill materials that affect the magnitude of the load transmitted to the pipe and the ability of the pipe to carry the load are at most important. In an urban setting, most public utilities and pipe culverts are installed in an open trench, it is important to

Understand the characteristics and how they affect the structural capacity of a rigid Concrete pipe buried under embankment. In the drainage design manual (2002), published by Ethiopian Road Authority (ERA) establishing drainage design manual, standards and analysis of pipe culverts has given emphasis for protection of the road through the prevention of damage due to erosion to achieve a chosen level of service without major rehabilitation at the end of a selected design period as economical as possible.

The design procedures considered that were taken into account the factors such as rainfall intensity, catchment areas, ground cover, and run-off. The procedures cover a range of drainage design applications currently used in Ethiopia. It was not clearly indicated the role of factors like: trench width and quality of contact between the pipe and bedding, pipe strength, bedding type, magnitude of lateral pressure, the minimum and maximum fill embankment height. In addition, type of back fill materials playing great role in proper functioning of the pipe culvert and its durability has not well classified.

Because of increasing number of heavy and light traffic flow in the cities at a fastest rate, the constructed roads become under high loading stress. This influences the load bearing capacity of pipe culverts that are located across and along the embankment of the road in the town. To increase durability of culverts, parameters to be considered include: trench width and quality of contact between the pipe and bedding, pipe strength, bedding type, magnitude of lateral pressure, and the minimum and maximum fill embankment. Therefore, to minimize this problem and make the serviceability of pipe culvert more intensive, minimum and maximum fill embankment height over the top of the pipe should be evaluated to meet the target. Pipe installation data has been taken from Ethiopian road corporation Construction and site visit in areas where existing drainages pipes that serve for a long period as well as those constructed recently in Jimma town. Sample data for back fill purpose has been collected from the site and laboratory tests for analysis input has been conducted. These tests are compaction test, Atterberg limit tests, grading tests, sieve analysis and hydrometer tests.

Further analysis has been accomplished using pipe Pac software for three edge bearing (3EB) Analysis. Calculation of earth loads and pipe classes for concrete pipe and determine appropriate pipe classes for specific back fill materials.

1. 2 Statement problem

In Ethiopia, most highway construction of asphalt and gravel road project, no matter how the number and length of drainage structure varies, it requires proper depth and installation of pipe culvert across and along the side of road under wearing surface of an embankment. The serviceability of pipe culvert is very important for the safe traffic flow by protecting the damaging effect of road materials by flood flow, erosion, or scouring which could shorten the serviceability as well as life span of pavement. At the project site where there is low standard installation of drainage structures, the problem will occur and disturbance of traffic flow may lead to deterioration of pavement, losing to government's budget that may not serve for the expected design life of the pavement.

The government spent millions of budget on the construction of highway project every year, but if there is no proper drainage structure provisions, it will be a loss of budget and causes great impact on economic development of the country.

The design and analysis system followed so far for pipe culvert in highway gives more emphasis for the rainfall intensity, catchment areas, ground cover, and run-off data as an input. However, important points has not been considered like: the minimum and maximum fill embankment height over the top of the pipe, trench width and quality of contact between the pipe and bedding, pipe strength, bedding type, magnitude of lateral pressure, axial thrust [12].

Therefore, to tackle the problem and to meet the objectives, different published research materials of which finite element analysis, experimental and standard design manuals has been assessed. Finally compared their output and identified the issues which will enable the researcher to choose appropriate design and analysis methods by the application of pipe Pac software package.

1. 3 Research Questions

- What was the maximum thickness cover for concrete pipe?
- What was the suitable back filling materials for concrete pipe cover?
- What are the possible types of loading on pipe surface?

❖ What are the factors to be incorporated in the design of concrete pipe that will help the designer?

1. 4 Objectives

1.4.1 General objectives

The main objective of the study is to evaluate the minimum and maximum fill embankment height over the top of the pipe culvert of drainage project

1.4.2 Specific objectives

1 To determine the minimum and maximum cover thickness for concrete pipe

2. To determine different loading conditions that influence the strength of buried pipe culverts under the wearing surface of road embankment.

3. To identify backfill material's property and differentiate factors that affect backfill material through conducting laboratory tests and compare with the available standards.

4. To create awareness for the designers in incorporating different loading types as a criteria in the design and analysis of concrete pipe.

1.5. Significance of the study

It will provide useful information for the designer to consider the criteria for design and analysis of concrete pipe culvert buried under embankment. Solve failure problem of pipe culvert, and it would create safe transportation for traffic flow.

On the other hand, it would save the budget by providing appropriate pipe structure which recognize three edge bearing strength analysis. Also it eliminates the scouring effect and runoff stagnancy on the outlet direction of pipe.

1.6 Scope of the study

The research addresses the general objectives and tries to identify the suitable backfilling materials for different strength of pipe culvert through collection of samples from three quarry sites around Jimma town. Two samples from each site has been collected and laboratory test has been done. And also determination of minimum and maximum cover thickness on top of concrete pipe culvert.

CHAPTER TWO

LITERATURE REVIEW

2.1 Background

During the first three decades of the 20th century, researchers at Iowa State University developed and tested a theory for estimating loads on buried pipe. Marston-Talbot advanced the original concept A. Marston (1930) continued the work on evaluation of design loads and published the Theory of External Loads on Closed Conduits or pipes in Light of the Latest Experiments, which presents the theory in its present form. During this same period, the three-edge bearing test was developed a method for evaluating the strength of rigid pipe. Other Iowa reports include Schlick tests of pipe on concrete cradles, and Spangler's classic report on the supporting strength of rigid pipe culverts, which still serves as the principal design theory [2].

In later work three bedding configurations and the concept of a bedding factor has been presented to relate the supporting Strength of buried pipe to the strength obtained in a three-edge bearing test. The theory proposed that the bedding factor for a particular pipeline and, consequently, the supporting strength of the buried pipe, is dependent on two installation characteristics: 1). Width and quality of contact between the pipe and bedding.2). Magnitude of lateral pressure and the portion of the vertical height of the pipe over which it acts [2].

For the embankment condition, he developed a general equation for the bedding factor, which partially included the effects of lateral pressure and for the trench condition establishment of conservative fixed bedding factors, which neglected the effects of lateral pressure, for each of the three beddings. This separate development of bedding factors for trench and embankment conditions resulted in the belief that lateral pressure becomes effective only at trench widths equal toor greater than the transition width. Such an assumption is not compatible with current engineering concepts and construction methods. It is reasonable to expect some lateral pressure to be effective at trench widths less than transition widths. Although conservative designs based on the work of Marston and Spangler have been developed and installed successfully for years, the design concepts have their limitations when applied to real world installations and these are:

- Loads considered acting only at the top of the pipe,
- Axial thrust were not considered.

- Bedding width of test installations less than width designated in his bedding configurations.
- Standard beddings developed to fit assumed theories for soil support rather than ease of and methods of construction.
- Bedding materials and compaction levels not adequately defined.

American Concrete Pipe Association (1970) began a long-range research program, on the interaction of buried concrete pipe and soil had done. The research resulted in the comprehensive finite element computer program, Soil-Pipe Interaction Design and Analysis (SPIDA), for the direct design of buried concrete pipe.

Since the early 1980's, SPIDA has been used for a variety of studies, including the development of four new Standard Installations, and a simplified microcomputer design program which were Standard Installations Direct Design (SIDD). The procedure presented here replaces the historical A, B, C, and D beddings used in the indirect design method that depend on the strength of the pipe.

The four Standard Installations table provide an optimum range of soil-pipe interaction characteristics. Type I Installation: allows relatively high quality materials and high compaction effort and it requires lower strength pipe, Type II: Allows silty granular soils with less compaction effort required for haunching and bedding. Type III: Allows use of soils with less stringent compaction requirements and Finally a Type 4 Installation requires a higher strength pipe, because it was developed for conditions of little or no control over materials or compaction.

Table2. 1Standard installation types and minimum compaction requirements [2].

Installation type	Bedding thickness	Haunch and outer bedding	Lower side
Type 1	Do/24 minimum, not less than 75 mm (3").If rock foundation, use Do/12 minimum, not less than 150 mm (6").	95% Category I	90% Category I,95% Category II, or 100% Category III
Type 2	Do/24 minimum, not less than 75 mm (3").If rock foundation, use Do/12 minimum, not less than 150 mm (6").	90% Category I or 95% Category II	85% Category I,90% Category II, or95% Category lIl
Type 3	Do/24 minimum, not less than 75 mm (3").If rock foundation, use Do/12 minimum, not less than 150 mm (6").	85% Category I, 90% Category II, or 95% Category III	85% Category I, 90% Category II, 95% Category III
Type 4	No bedding required, except if rock foundation, use Do/12 minimum, not less than 150 mm (6").	No compaction required, except if Category III, use 85% Category III	No compaction required, except if Category III, use 85% Category III

Source: American concrete pipe Association. www. Concrete pipe.org; 2011

Also description of generic soil type both in Unified soil classification (USCS) and American Association of State Highway and Transportation Officials (AASHTO) soil classifications equivalent to the generic soil types in the Standard Installations table (2.2).

Table2. 2 Equivalent USCS and AASHTO soil classification for SIDD soil designation [2]

SIDD Soil	Representative Soil Types		Percent Compaction	
	USCS,	Standard AASHTO	Standard Proctor	Modified Proctor
Gravelly Sand (Category I)	SW, SP, GW, GP	A1,A3	100 95	95 90
Sandy, Silt (Category II) sieve	GM, SM, ML, Also, GC ,SC with less than20% passing #200	A2, A4	100 95 90 85 80 49	95 90 85 80 75 46
Silty, Clay (Category III)	CL, MH, GC, SC	A5, A6	100 95 90 85 80 45	90 85 80 75 70 40

In1983, the indirect design method developed by Marston-Spangler was included in a new section of the American Association of State Highway and Transportation Officials Bridge Design Specifications (AASHTO).

Whether flexible or rigid pipe depends on the backfill structure to transfer loads to the bedding surface. Pipe must be installed as designed to perform as expected service time. Material properties, backfill criteria, and load conditions also govern the procedure. Minimum and Maximum burial depths can vary greatly depending on the application, product, backfill material, and compaction level.

Both flexible and rigid pipe depend on proper backfill. In the case of flexible pipe, deflection allow loads to transfer. Rigid pipe transmits most of the load through the pipe wall into the bedding. In both cases, proper backfill is very important in allowing this load transfer to occur [3].

Another research finds out that for positive projection, a conduit installed on a non-yielding foundation is considerably stiffer than the surrounding fill material. As a result, greater settlement will occur in the exterior prisms than within the interior prism. As the soil in the exterior prism moves downward relative to the interior prism, it exerts a down ward force due to the frictional nature of the backfill material. The resulting load on the conduit is equal to the weight of the overlying soil plus the frictional forces [8].

For negative projection, a conduit installed in a narrow trench beneath an embankment is defined as a negative projection installation. The frictional forces between the fill material and sides of the trench decrease the earth load on the conduit. The earth load on the conduit equals the weight of the overlying soil less the frictional forces. Additional sub-category based upon frictional forces within the backfill material. If the magnitude of relative settlement between the prisms is sufficient that the frictional forces extend to the surface of the fill, the pipe can be defined as in a complete condition. In opposite to this, if the frictional forces cease to exist at an imaginary horizontal plane within the fill, the pipe is defined an incomplete condition [8].

The imperfect ditch or induced trench conduit uses a concept which is the same as that of a negative projecting conduit, but in this case trenches are cut into the embankment over the conduit and backfilled with compressible material. This type of installation is effective for reducing backfill load on a pipe, but cannot be used in embankments that serve as water barriers because the loosely placed backfill will admit channeling of seepage water through the embankment [2].

A ditch conduit was defined as one that is installed in a relatively narrow ditch and covered with earth backfill. Trench used in relatively narrow excavations, and the pipeline covered with earth backfill, which extends to the original ground surface as shown in figure (1) below. The trench load based upon certain applied mechanics assumptions concerning the properties of the materials involved and these assumptions were earth loads on the pipe develop as the backfill settles [2].

The resulting earth load on the pipe is equal to the weight of the material in the trench above the top of the pipe minus the shearing (frictional) forces on the sides of the trench, cohesion was negligible because with cohesive soils, considerable time must elapse before effective cohesion between the backfill material and the sides of the trench can develop.

Therefore, the assumption is no cohesion, which yields the maximum probable load on the pipe, and for a rigid pipe, the side fills may be relatively compressible and the pipe will carry a large portion of the load developed over the entire width of the trench. Active lateral pressure against the pipe is neglecting, but it should be taken into account if the trench width exceeds the defined narrow trench widths. The type of bedding is one of the factors that determine the supporting strength of buried pipe. Types of bedding for the trench condition are shown in Figure (2.1) and Table (2.1).

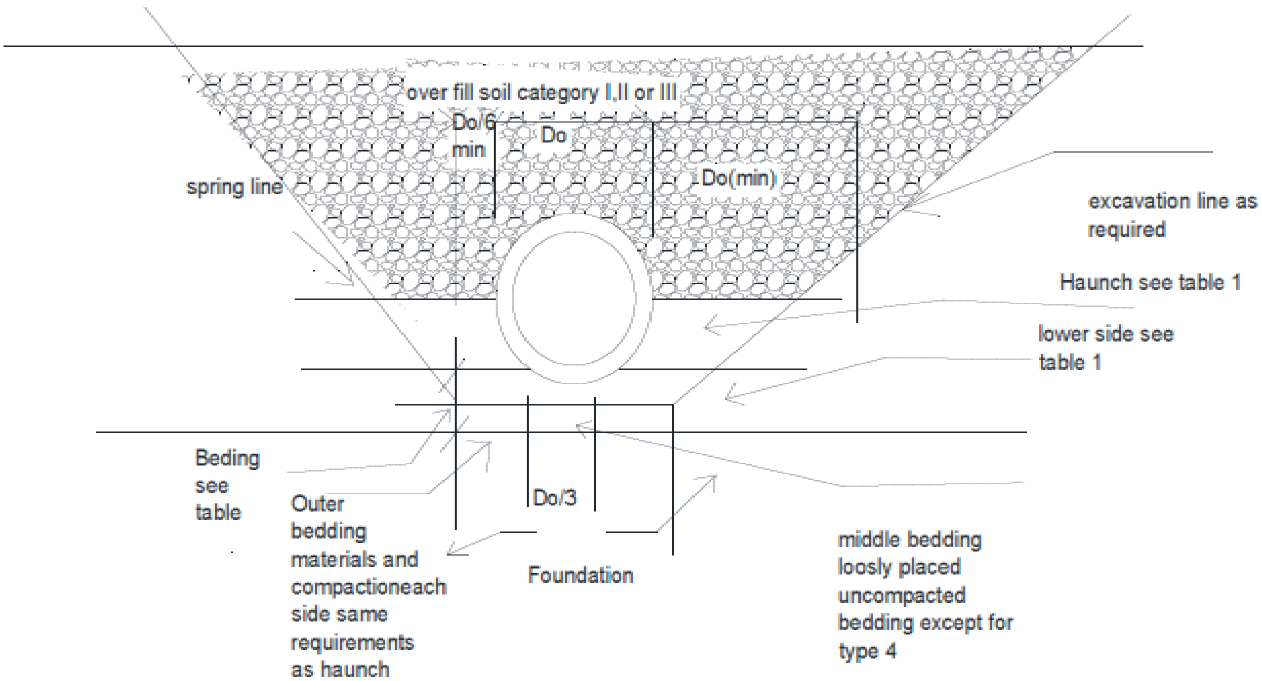


Figure 2.1Standard Trench Installation [15]

2.2 General requirements for Installation of Buried pipe

2.2.1 Pipe Soil Interaction Approach

The structural performance of pipe depends on the interaction between the embedment or backfill envelope, and the pipe, and commonly refers as pipe/soil interaction. The backfill envelope must provide structural and drainage characteristics appropriate for the application. Structural considerations of the backfill include the type of material and compaction level, dimensions of the backfill envelope, and native soil conditions. The type of material (sand, gravel, clay, etc.) and compaction level (standard Proctor density) determine overall strength of the backfill. Generally, material particles that are relatively large and angular require less compaction than particles that are smaller and less angular to produce structures having equal strength [3].

2.2.2 Backfill Material and Compaction Mechanism

Mechanical compaction is not always necessary; dumping of some backfill materials and others can meet minimum compaction criteria simply by walk in around the pipe. On the other hand, mechanical compaction can make placement of some backfill materials much faster.

Another backfill material that has gained in application over the past few years is flow able fill. This material is similar to a very low strength concrete and it is poured around the pipe and hardens to form a solid backfill structure. The final cured strength of this material is highly dependent on mix design. In order to take advantage of the strength of this material, the backfill strength of the surrounding native material must be adequate. Proper compaction of the approach sections is essential in order to provide a smooth and uniform running surface across the culvert pipe. If the embankment is not properly compacted before and after the culver, it will continue to settle after traffic is allowing on the road. The culvert pipe is a rigid structure so the section above the culvert pipe will be subject to less settlement as compared with the adjoining sections. As a result, the traffic will cause more consolidation of the road body before and after the culvert, and the road section directly above the culvert pipe will appear as a bump in the road surface. However, with proper compaction, this potential defect can be avoided altogether [3, 19]

2.2.3 Cover Heights

Sarah L. and Gasman (2005), indicates the minimum fill height for all types of pipe is measured from the top of the pipe to the top of soil backfill. All pipes should meet minimum cover requirements and should not use under roadways when these minimum cover heights cannot achieve. In some driveway applications, it may be difficult to achieve minimum cover. Sometimes to tackle the problem Concrete elliptical pipe and aluminum pipe arch are good alternatives to circular pipe when additional room for cover needs. A greater minimum fill height is required on top of pipe culverts to prevent damage to the pipe from loads induced by heavy construction equipment. Therefore, no heavy equipment shall be driven over any pipe culvert until the backfill is completed to the minimum allowable cover height for construction loading as presented in the "South Carolina, Department of Transportation (SCDOT) Culvert Pipe Selection Guide" so that damage does not occur to the pipe. It was recommended that minimum cover must maintained until heavy equipment usage discontinued [19].

Table2. 3Cover Height for circular reinforced concrete pipe [19].

Installation Type	Pipe Diameter, mm	Maximum Height of Fill (m)			Minimum Allowable Cover Height (m)	
		Class III AASHTO M170	Class IV AASHTO M170	Class V AASHTO M170	HS-20 Vehicle Loading	Construction Vehicle Loading.
Type I	300-900	8.4	12.4	12.4	0.3	0.9
	105-165	8	12	18	0.3	0.9
	180-240	7.8	12	17.7	0.3	0.9
Type II	300-750	5.9	8.7	13	0.3	0.9
	900-240	5.5	8.4	12.7	0.3	0.9
Type III	300-1000	4.3	6.5	10	0.3	0.9
	1200-2400	4	6.5	10	0.3	0.9
Type IV	300-525	9	4.3	6.5	0.3	0.9
	600-2400	9	4.6	7	0.3	0.9

FromTable2.3the specification for installation type is expressed as per ASTM C 1479 and American Association of State Highway and Transportation Officials, Washington D.C., 2002.

(AASHTO) Section 27, Standard Specification for Highway Bridges, Division II: Construction, Maximum fill heights is based on American Concrete Pipe Association (ACPA) Charts.

2.2.4 Soil classification

Sarah L. and Gasman (2005), specifies the methodology how Soils are commonly classified using the Unified Soil Classification System (ASTM D 2487) or the AASHTO Soil Classification System (AASHTO M 145). In addition, ASTM D2321 divides the soils into different "Classes." Therefore the equivalent ASTM and AASHTO Soil Classifications is shown in Table (4).

Table2. 4.The equivalent ASTM and AASHTO Soil Classifications [19].

Basic Soil Type	ASTM D 2487	AASHTO M 145	ASTM D 2321
Sn(Gravely sand)	SW, SP, GW, GP sands and gravels with 12% or less fines	A-1, A-3	Class IB: Manufactured, processed aggregates; dense graded, clean Class II: Coarse-grained soils, clean
Si (sand silt)	GM, SM, ML Also GC and SC with less than 20% passing a No. 200 sieve	A-2-4, A-2-5, A4	Class III: Coarse-grained soils with fines ClassIVA: Fine-grained soils with no to low plasticity
Cl (silty clay)	CL, MH, GC, SC Also GC and SC with more than 20% passing a No. 200 sieve	A-2-6, A-2-7, A-5, A-6	Class IVA: Fine-grained soils with low to medium plasticity

From the above standards, the use of sands and gravels for the structural backfill (bedding, haunch and embedment) will provide the greatest assurance of good performance. Sands and gravels without fines achieve good densities when dumped and excellent densities when compacted. If placed, spread and compacted in moderate lift thicknesses, excellent pipe support is ensured for all typical installations. The materials provide excellent pipe performance when placed and compacted and are less sensitive to poor construction practices than other materials [19].

2.2.5 Installation Inspections

Sarah L. and Gasman (2005), states that during construction, the trench width, bedding, backfill, soil type or soil density, and fill height must be checked to ensure that they meet the specifications to ensure a proper installation. The pipe and joints must laid according to the engineering drawings and specifications. The pipe and joints must be inspected to ensure that they are sealed and soil tight. The bedding and backfill materials must be inspected to certify whether that they meet specification or not and sufficient quantities are available to backfill the pipe. Compaction and density tests must be performed at every stage of construction to ensure that the soil is compacted to the appropriate level. In addition, specification shall be furnished with quality control data from the contractor to make sure that compaction requirements have met. The thicknesses of the bedding, backfill and cover layers must be measured and checked against specifications [19].

2.3 Foundation soil property

The trench foundation provides the base for the bedding material and must provide uniform, stable support for the pipe. Soils for the foundation may consist of the native soil or a modification. Organic material or soft or low density soil is not suitable because it can cause differential settlement. Very soft, wet soils should be replaced or reinforced by working in drier or stronger soil and compacting well [19].

2.4 Design procedure for the selection of pipe strength

2.4.1 Effects of loads on buried pipes

Buried pipe under embankment serve as two functions, hydraulically and structural function. It must provide a passage for the fluid that it is designed and it also must fit the bedding surface to support the weight of the ground and any load applied on it .Unless pipe is installed properly it will cause series damage on road pavement and other structure that is found nearby. There are different types of loads applied on reinforced concrete pipe which it must resist or carry the weight resulting from dead load of over burden pressure and in addition to this any live load and static loading [17].

2.4.2 Determination of Earth Load

Embankment Soil Load: The type of installation has a significant effect on the loads carried by the rigid pipe (concrete pipe). Although narrow trench installations are most typical, there are many cases where the pipe is installing in a positive projecting embankment condition, or a trench with a width significant enough that it should be considered a positive projecting embankment condition. In this condition, the soil alongside the pipe will settle more than the soil above the rigid pipe structure, thereby imposing additional load to the prism of soil directly above the pipe. As fill height increase there is a chance of dead load problem in which the weight of the soil supported by a pipe is increasing in similar manner. With the Standard Installations, this additional load should be accounted for by using a Vertical Arching Factor (VAF) and then this factor is multiplied by the prism load or weight of soil directly above the pipe (PL,) to give the total load of soil on the pipe. American Concrete Pipe Association [15].

Different formulas were presented by publisher given in Table below for calculating earth load.

Table2. 5Summary of Earth load determination formula suggested by different researcher

Formulas	Expression	Publishers
$We = VAFxPL$	Un-factored earth load	American Concrete Pipe Association
$PL = w + H\frac{Do(4-\pi)}{8}Do$	Prism load.	
W_e = PL x VAF	Un-factored earth load	Marston-Spangler theory
$PL\ = \left(\frac{WDo}{12}\right)(H + \left(\frac{0.107Do}{12}\right)$	Prism load.	
$We\ = FewBeH$	Un-factored earth load	AASHTO-LRFD Specifications(2000)
$D - load = \left[\frac{W_L}{B_{fLL}} + \frac{W_E}{B_{fe}}\right]F.S$	D- Load	Ontario concrete pipe Association
$W_E =\ C_d\, wg\, B_d^2$	Trench Backfill Load	

Where: w = soil unit weight We = un-factored earth load

H = height of Arching Factor of fill B_e = out-to-out horizontal dimension of pipe,

Do = outside diameter, VAF = Vertical arching factor and

PL = prism load. Bd = width of trench at top of pipe, meter

$BfLL$ = Live load bedding factor Bfe = embankment bedding factor

WE = Trench back fill load g = gravitational constant

WL = $\frac{W_T}{Le}$ is live load on pipe (the ratio of total live load to effective supporting length)

$WT = W_L LS_L$ (Total live load),

WE = Trench back fill load

WL = $\frac{W_T}{Le}$ is live load on pipe (the ratio of total live load to effective supporting length)

$WT = W_L LS_L$ (Total live load),

$L_e = L + 1.75(0.75Bc)$ is effective supporting length of pipe

F_e = soil-structure interaction factor for the specification Installation

L = Length of ALL parallel to longitudinal axis of pipe

$$C_d = \frac{1- e^{-2K\mu'\frac{H}{Bd}}}{-2K\mu'}$$, Trench load coefficient

K = Lateral pressure ratio for back fill or backfill material

e = Base of natural logarithm (2.178)

μ' = Coefficient of sliding friction between the backfill materials and trench

From the above empirical formulas the third simplified formula for calculating the earth load which was given in AASHTO-LRFD Specifications is tabulated in table (3). According to Dr. Frank J. Heger had stated that an evaluation of the output produced a load pressure diagram significantly different than proposed by previous theories.

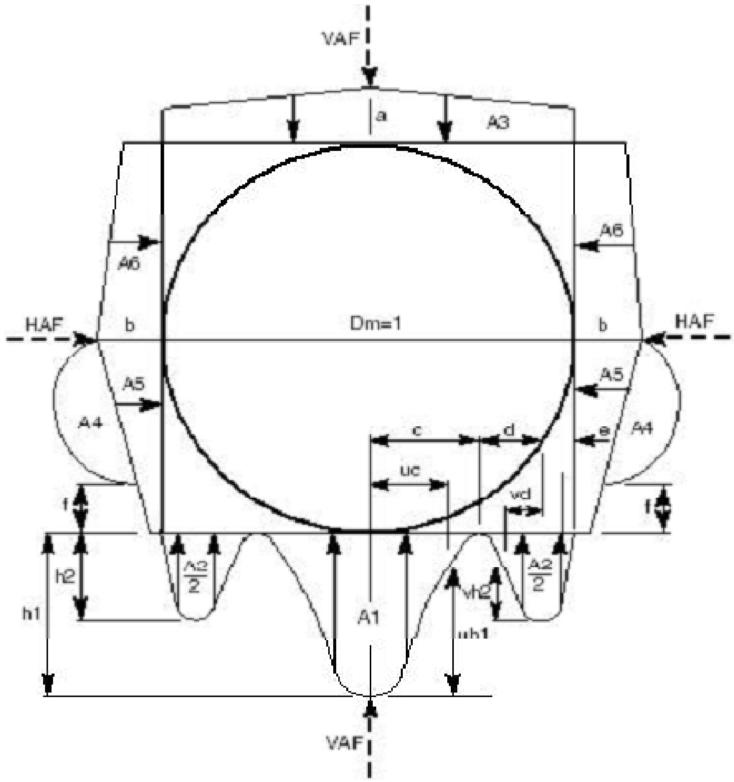


Figure 2.2Heger Pressure Distribution [12]

Table2. 6Coefficients and Arching Factors for each installation type.

Installation type	1	2	3	4
VAF	1.35	1.4	1.4	1.45
HAF	0.45	0.4	0.37	0.3
A1	0.62	0.85	1.05	1.45
A2	0.73	0.55	0.35	0
A3	1.35	1.4	1.4	1.45
A4	0.19	0.15	0.1	0
A4	0.08	0.08	0.1	0.11
A6	0.18	0.17	0.17	0.19
a	1.4	1.45	1.45	1.45
b	0.4	0.4	0.4	0.3
c	0.18	0.19	0.2	0.25
e	0.08	0.1	0.12	0
f	0.05	0.05	0.05	0
u	0.82	0.82	0.85	0.9
v	0.8	0.7	0.6	0

For Standard Installations the earth pressure distribution shall be the Heger pressure distribution shown in Figure (2) for each type of Standard Installation.

The above pressure distribution and arching factors figure were briefly discussed in the following manner in which their designation for different expressions and how they can define in specific site. VAF and HAF are vertical and horizontal arching factors, these coefficients are representing non-dimensional total vertical and horizontal loads on the pipe respectively. The actual total vertical and horizontal loads are (VAF) X (PL) and (HAF) X (PL), respectively, where PL is the prism load.

Coefficients A1 through A6 represent the integration of non-dimensional vertical and horizontal components of soil pressure under the indicated portions of the component pressure diagrams (i.e., the area under the component pressure diagrams). The pressures are assumed to vary either parabolic or linearly, as shown, with the non-dimensional magnitudes at governing points represented by h1, h2, uh1, vh1, a and b. Non-dimensional horizontal and vertical dimensions of component pressure regions are defined by c, d, e, uc, vd and f coefficients. Where d, h1, and h2 are calculated as follows.

$d = (0.5c - e)$, $h1 = \frac{1.5A1}{(c)*(1+u)}$, $h2 = \frac{1.5A2}{[(d)(1+v)+(2e)]}$

Trench Soil Load: In narrow or moderate trench width conditions, the resulting earth load is equal to the weight of the soil within the trench minus the shearing (frictional) forces on the sides of the trench. Since the new installed backfill material will settle more than the existing soil on the sides of the trench, the friction along the trench walls will relieve the pipe of some of its soil burden. The Vertical Arching Factors in this case will be less than those used for embankment design [12].

2.4.2.1 Pipe Weight

As per Edmonton design and construction standards pipe weight may not be a significant component of load relative to other loads in buried pipe analysis. Because it is already accounted for in a three-edge bearing test that it can be ignored in accounting for overall loads in analysis [1]

The approximate weight of circular pipe is given by

$W_p = 3.3h(D_i + h)$ ------------------------------ [11]

The wall thickness for circular pipe is often referred to in standard designation of “A”, “B”, or “C” wall thicknesses. The relationship between wall thickness, wall thickness type and inside diameter is governed by the following expressions.

Wall A, $h = D_{\frac{i}{12}}$

Wall B, $h = (\frac{Di}{12} + 1)$,

Wall C, $h = (\frac{Di}{12} + 1.75)$ ---- [11]

Where: h = wall thickness

Di = inside diameter

W_p = weight of circular pipe

2.4.2.2 Settlement Behavior

To evaluate the height of the plane of equal settlement above top of pipe (He), figure (2) it is necessary to determine, numerically, the relationship between the pipe deflection and the relative settlement between the prism of fill directly above the pipe, and the adjacent soil and this relationship is settlement ratio, which expresses as settlement Ratio for Positive Projecting Embankment.

Settlement, which affects loads on negative projecting embankment installations, is indicating in Figure 3. As in the case of the positive projecting embankment installation, it is necessary to find out the settlement ratio, by relating the deflection of the pipe and the total settlement of the prism of fill above the pipe, to the settlement of the adjacent soil. This relationship is defined as a settlement ratio [20].

Settlement Ratio for Positive Projecting Embankment,

$$r_{sd} = \frac{(s_d + s_g) - (s_f + d_c)}{s_m}, \quad \text{--------------------------------[18]}$$

Settlement Ratio for Negative Projecting Embankment,

$$r_{sd} = \left(1 - \left(\frac{s_d + s_f + d_c}{s_g}\right)\right), \quad \text{------------------------------ [18]}$$

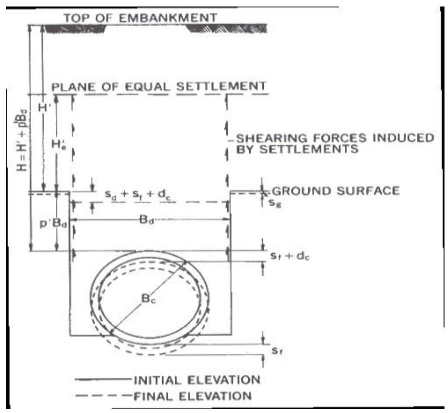
Where: Sg = settlement of the natural ground or compacted fill surface adjacent to the pipe

Sm = settlement of the adjacent soil of height

Sf = settlement of the pipe into its bedding foundation

dc = deflection of the vertical height of the pipe

Sd = stand or compression of the fill material in the trench within the height for positive and negative projecting embankment installations.



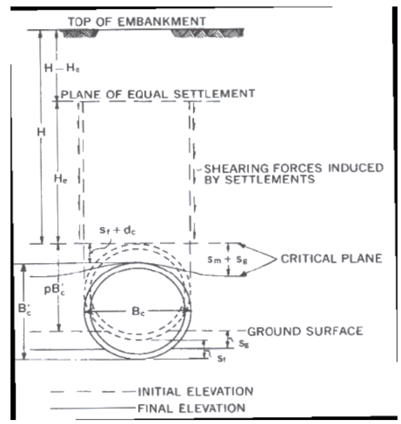


Figure 2.3Settlements Which Influence Loads Positive Projecting Embankment Installation [18]

Figure 2.4Settlements Which Influence Loads negative projecting Embankment Installation [18]

2.4.3 Determination of Live Load

Live loads are loads due to traffic movement or flow over the installed pipe and this load is applied to a certain area of the surface, contact area of the tire. As depth of installation of pipe increase downward, it is subjected to a lower intensity of loading from surface load than a shallow covered load. Design table for maximum Allowable soil cover often include a surcharge load to represent traffic or construction loadings

In the selection of pipe, it is crucial to evaluate the effect of live loads, it's considerations are necessary in the design of pipe installed with shallow cover under surfaced and unsurfaced highways. The distribution of a live load at the surface on any horizontal plane in the subsoil is shown in Figure (3). The intensity of the load on any plane in the soil mass is greatest at the vertical axis directly beneath the point of application, and decreases in all directions outward from the center of application. As stated above if the distance between the plane and the surface increases, the intensity of the load at any point on the plane decreases [18].

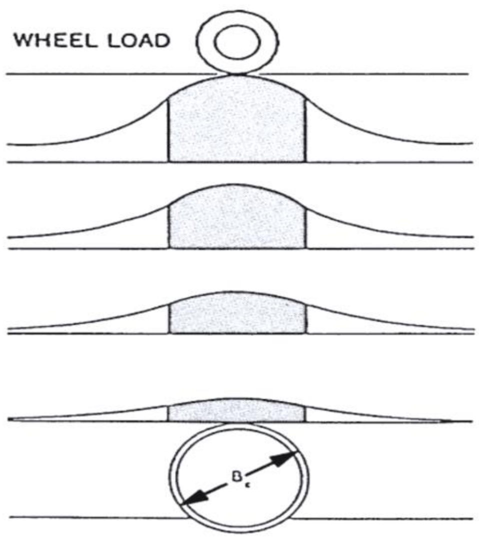


Figure 2.5Live Load Distributions on the pipe [15]

2.4.3.1 Truck and Traffic Loads –AASHTO method

According to simplified AASHTO method can be used to estimate concentrated wheel loads for either AASHTO series vehicles or standard vehicle configurations conforming to the CL series trucks as set out in the CAN/CSA –S6-00 Canadian Highway Bridge Design Code (CHBDC). The CL- W series truck, for example, is a simplified five- axle vehicle for which the W indicates the total gross vehicles load in KN as set out in the CAN/CSA-S6-00 Canadian Highway Bridge design Code (CHBDC). A CL-65 design vehicle would therefore have a gross vehicle weight of 625kN. The load is distributed over both sets of dual tires (each .60m x 0.25m), at approximately 1.80m center to center. The per-axle load distribution for CL-W series trucks is shown in Figure 14 from the CHBDC [11].

Some of typical design vehicle series according to Canadian Highway Bridge Design Code is given CL – 625, CL -750, CL-800, and CL-850.The AASHTO H and HS series design vehicle represents a simplified or idealized five-axle truck. In this case the associated load is given for the single axle carrying the largest load. The following table lists some typical AASHTO design vehicles and their associated loads.

Design vehicle	Front(lb)	Rear(lb)	
H- 25	10,000	40,000	
H- 20	8,000	32,000	
H- 15	6,000	24,000	
H -10	4,000	16,000	

Design vehicle	Front(lb)	Rear(lb)	Rear(lb)
HS- 25	10,000	40,000	40,000
HS- 20	8,000	32,000	32,000
HS- 15	6,000	24,000	24,000

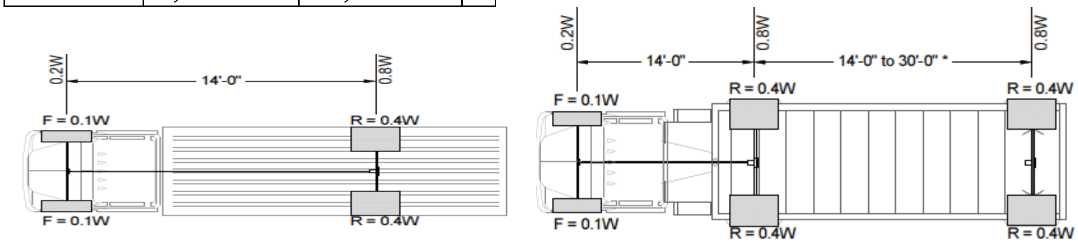
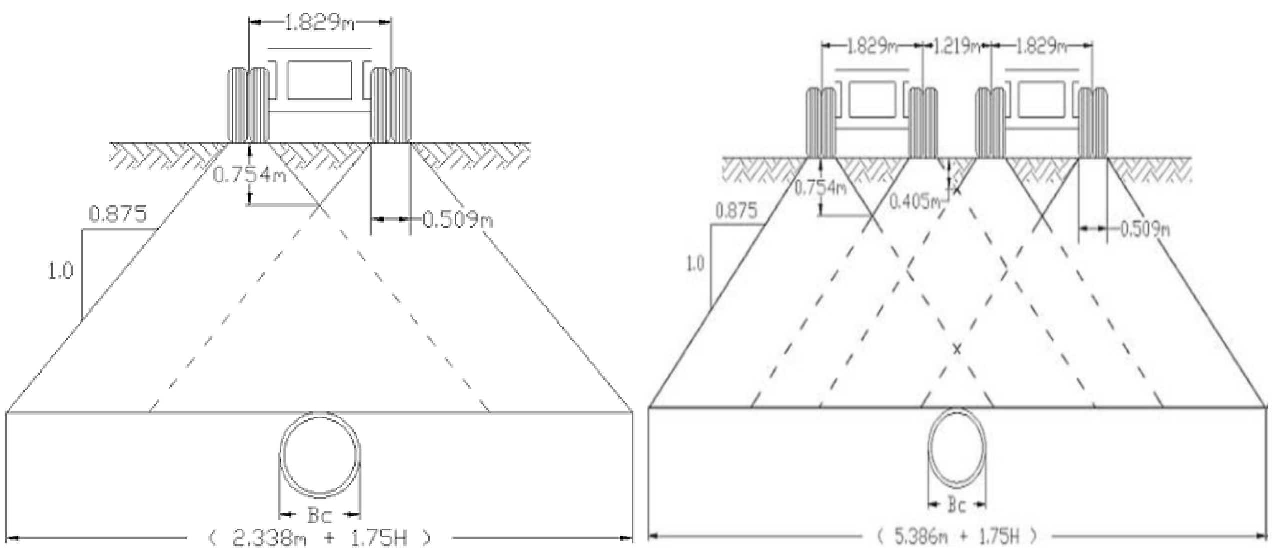


Figure 2.6AASHTO highway Loads [11]

In the AASHTO simplified live load method the load for a single axle is considered to be distributed over dual tires with a total contact area of 0.25m x 0.51m spaced at approximately 1.83m. The load is assumed to increase with depth in a pyramidal fashion as depicted in Figure (7).

Figure 2.7- Zones of Influence and Impact Factors at Depth [11]



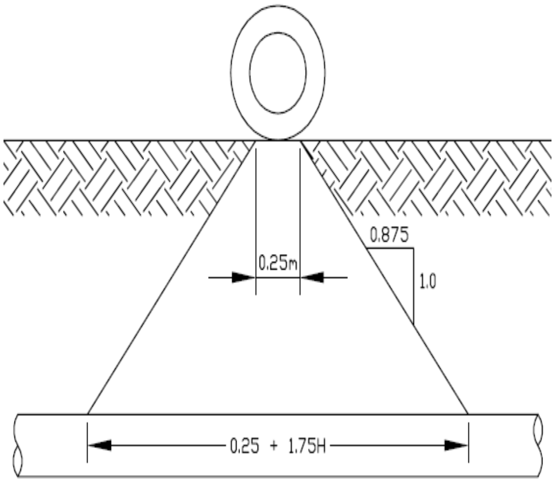
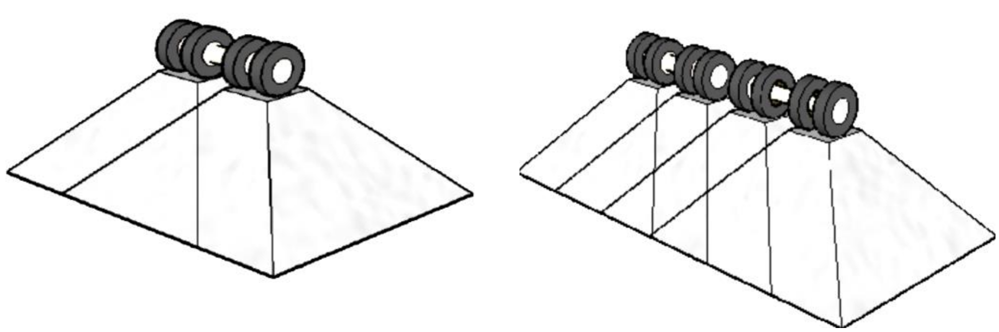


Figure 2.8Ameron Concrete Cylinder Pipe Design Manual 1988[11]

Impact Factor	
cover(m)	If
0.3	0.5
0.61	0.5
0.76	0.43
0.91	0.38
1.07	0.3
1.22	0.23
1.37	0.17
1.52	0.1
1.68	0.04
1.75	0

At a depth of 0.75m the influence areas overlap and the total load from both sets of tires is assumed to be evenly distributed over the entire area. Thus, for depths less than 0.75m, the single axle load can be divided by two. For depths greater than 0.75m, the pressure can be calculated as noted in figure (9&10).



$wL = \frac{single\ axle\ load}{(2.34+1.75H)(0.25+1.75H)}$, SI units $\quad wL = \frac{Dual\ axle\ load}{(5.39+1.75H)(0.25+1.75H)}\ SI\ units$

Figure 2.9- AASHTO Method for Dual Passing Vehicles [11].

Where, H is the depth below the surface at which the load is to be estimated.

AASHTO method specify that Once the pressure per unit length wL has been determined, the total live load WLhas been converted to pipe load units consistent with the load per unit length format identified for earth loads and include the effects of impact loads (impact factor) and this expression is then given by,

$\mathrm{W}L \;=\; wLBc\;(1\;+\;If)$ ------------------------------ [11]

Where: WL = Total live load

wL = pressure per unit length

Bc = outside horizontal span of the pipe, meters

If = impact factor

2.4.4 Selection of Bedding

Bedding under the pipe culvert is provided to distribute the vertical reaction around the lower exterior surface of the pipe, and to reduce stress concentrations within the pipe wall. The load that a concrete pipe will support depends on the width of the bedding contact area, and the quality of the contact between the pipe and bedding.

For every types of bedding to be used, the center third of the bedding is to remain uncompact for pipe settlement and initiation of haunch support. An important consideration in selecting a material for bedding is to be sure that positive contact can be obtained between the bed and the pipe. Since most granular materials will shift to attain positive contact as the pipe settles, an ideal load distribution can be attained through the use of clean coarse sand, or well-graded crushed stone. To ensure that the in-place supporting strength of the pipe is adequate, the width of the band of contact between the pipe and the bedding material should be in accordance with the specified class of bedding. With the development of mechanical methods for sub grade preparation, pipe installation, backfilling and compaction, the flat bottom trench with granular foundation is generally the more practical method of bedding. If the pipe is installed in a flat bottom trench, it is essential that the bedding material, directly under the pipe, be loosely compacted over a width equal to one third of the outside diameter of the pipe, and be uniformly compacted under the haunches of the pipe[18].

2.4.5 Determination of Bedding Factor

The bedding factor is the ratio of the strength of pipe, under the installed conditions of loading and bedding, to the strength of the pipe in the plant test. Spangler as the load factor defined this same ratio originally. This latter term, however, was subsequently defined in the ultimate strength method of reinforced concrete design, with an entirely different meaning. To avoid confusion, therefore, Spangler's term was renamed the bedding factor. The three-edge bearing test shown in Figure 5 is the normally accepted plant test; all bedding factors described relate the in-place supporting strength to the three-edge bearing strength [15].

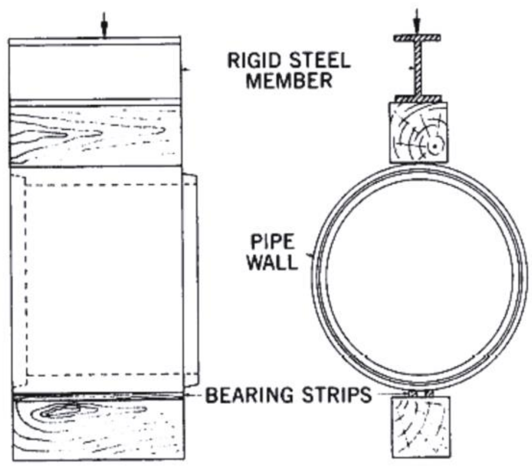


Figure 2.10Three-Edge Bearing Test [15]

The required three-edge bearing strength of circular reinforced concrete pipeis expressed as D-load and is computed by the equation:

$$T.E.B = \left[\frac{W_L+W_E}{B_f}\right]FS \text{ --------------------------- [2]}$$

Where: $T.E.B$ = Three edge bearing strength

W_L= Total live load on pipe

W_E = Total trench backfill load

B_f = bedding factor

$F.S$ = Factor of safety

Table2. 7Bedding Factors for Circular Pipe of different bedding types [2]

pipe inside diameter(in)	Type 1	Type2	Type3	Type4
12	4.4	3.2	2.5	1.7
24	4.2	3	2.4	1.7
36	4	2.9	2.3	1.7
72	3.8	2.8	2.2	1.7
144	3.6	2.8	22	1.7

Note: For pipe diameters other than listed, embankment condition bedding factors, Bfe can be obtained by interpolation. Bedding factors are based on soils being placed with the minimum compaction specified in Table (1) for each AASHTO Standard Installation.

2.4.5.1 Trench Bedding Factors

The two researchers Spangler and Schlick postulated that some active lateral pressure is developed in trench installations, before the transition width is reached. As the trench width increased for a given height of cover and pipe diameter, a point is reaching at which no additional load is transmitting to the pipe, and an embankment condition applies. This limiting value of the trench width is defined as the transition width. Experience indicates that the active lateral pressure increases as the trench width increases, from a very narrow width to the transition width, provided the side fill is compacted. Defining the narrow trench width as a trench having a width at the top of the pipe equal to or less than the outside horizontal span plus 300 mm, and assuming a conservative linear variation between this narrow trench width and the transition width, the variable trench bedding factor can be determined[15].

2.4.6 Application of factor of safety

The total earth and live load on a buried concrete pipe is computing and multiplying by a factor of safety to determine the pipe supporting strength required. The safety factor is defined as the relationship between the ultimate strength D-load (D_{ult}) and the 0.3 mm crack D-load (D0.3). This relationship is specified in the Canadian Standards Association (CSA) standards on reinforced concrete pipe. Therefore, for reinforced concrete pipe, a factor of safety of 1.0 will apply if the 0.3 mm crack strength is used as the design criterion. For non-reinforced concrete pipe, a factor of safety of 1.25 to 1.5 is normally used [15].

$$D - load = \left[\frac{W_L+W_E}{B_f D}\right] FS \text{ ----------------- [15]}$$

Where: W_L = Total live load on pipe

W_E = Total trench backfill load

B_f = bedding factor

$F.S$ = Factor of safety

2.4.7 Selection of Pipe Strength

The Canadian Standards Association (CSA) and the American Society for Testing and Materials (ASTM) have developed standard specifications for precast concrete pipe. Each specification contains design, manufacturing and testing criteria. CSA-A257.1-M92 for circular concrete culvert, storm drain and sewer pipe specifies three strength classes for non-reinforced concrete pipe. These classes are specified to meet minimum ultimate loads, expressed in terms of three edge-bearing strengths in kilonewtons per linear meter.

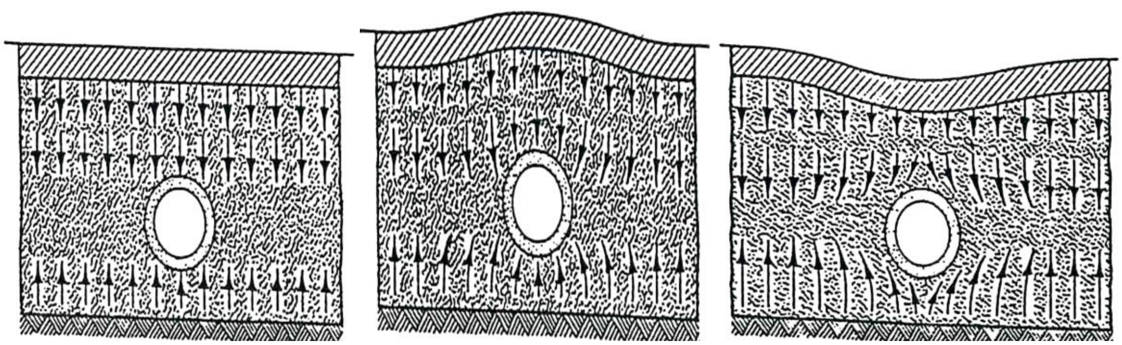
CSA-A257.2-M92 for circular reinforced concrete culvert, storm drain and sewer pipe specifies strength classes based on D-load at 0.3 mm crack ($D_{0.3}$) and ultimate load (D_{ult}). The 0.3 mm crack D-load ($D_{0.3}$) is the maximum three-edge-bearing test load supported by a concrete pipe, before a crack occurs having a width of 0.3 mm measured at close intervals, throughout a length of at least 300 mm. The ultimate D-load (D_{ult}) is the maximum three-edge-bearing test load supported by a pipe. D-loads is expressed to be in Newton's per linear meter per millimeter of inside diameter. In other way expression Ultimate D-load states as the required D-load at which the pipe develops its ultimate strength in a three-edge-bearing test is the design D-load (at 0.01-inch crack) multiplied by a strength factor that is specified in AASHTO materials specifications M 170 or M 242 (ASTM C 76 or C 655) for Circular pipe [10].

Table2. 8. D-load specification for Reinforced concrete pipe [10]

class	To produce a 0.3mm crack		Ultimate load	
	D-load	F.S	D-load	F.S
40-D	40	1	60	1.5
50-D	50	1	75	1.5
65-D	65	1	100	1.5
100-D	100	1	150	1.5
140-D	140	1	175	1.25

2.5 Transmission of load

Loads applied to a soil mass are transmitted downward through it along a regular, smoothly flowing paths or lines. Broad load such as embankment, applied over wide areas are transmitted vertically down ward along parallel paths with slowly diminishing intensities. As shown in figure (7) when it is necessary to place a pipe in continuous soil, it will receive what might be accounted as its proper share of the load only if it does not significantly change the pattern of load distribution within the soil medium figure (A). Pipe which is more rigid than the surrounding soil will stiffly accept more than its fair share of the load and cause the soil beside the pipe to be less heavily loaded figure (B). A pipe able to compress more than the surrounding soil will yield or shed some of the superimposed load to the soil beside it, figure (C)



A .Pipe supporting its proper of the load B. pipe supporting more than its Share of the load C. pipe supporting less than its share share of the load

Figure 2.11. Effect of Flexibility of Pipe on Supporting Ability [8]

CHAPTER THREE

MATERIALS AND METHODS

3.1 Location of study area

Jimma is located 353kms southwest of Addis Ababa and it is a special zone of the Oromia Region. It has a latitude and longitude of 7°40′N 36°50′E and elevation of 1,780m above sea level, Annual rainfall is one of the highest in the country receiving 1200-1700mm per year. The main rainy season, lasts from April to October. Temperatures are moderate with highest of 25-30 ^{0}C and lowest of 7-12 ^{0}C. The topography of the zone is mountainous and highly covered with forest. The population is moderately dense with a total number of 207,573. The dominant agro ecology zone is midlands. Wild animals and dense forest of indigenous trees eucalyptus are the other natural resources [13].

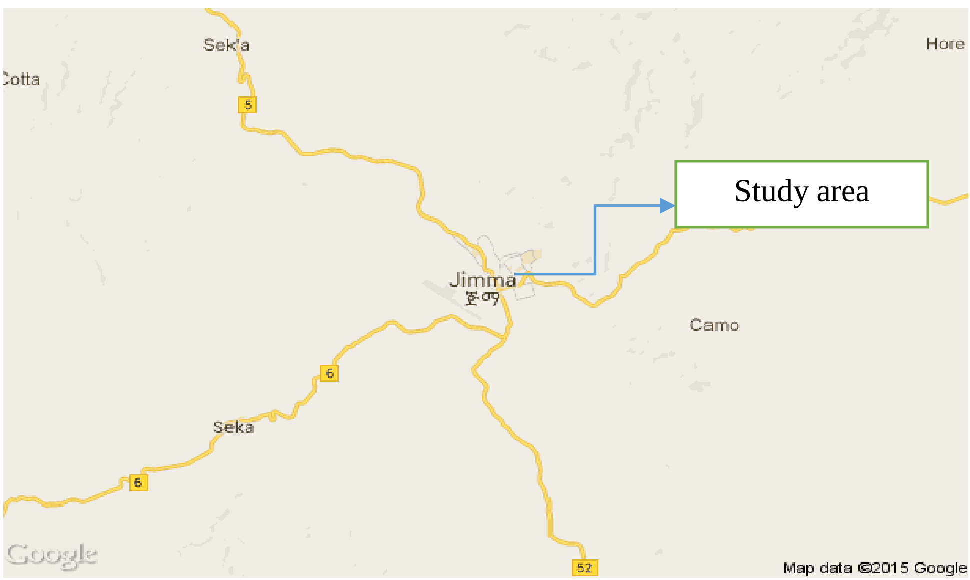


Figure 3.1Study area location

3.2 Source of materials

There are about five quarry locations for the source of backfill materials for construction of road from which three of them were identified and considered for study because they are used as a backfill materials in the Jimma town. The existing situation by which drainage of reinforced concrete pipe installed across and along the highway pavement road in different locations of the

City .The following photo were taken from different sites of road, showing some certain structure failures due to insufficient cover thickness of backfill and completely uncovered.

Figure 3.2Photos of Existing Drainage around Bus Station

Failure of pipe culvert was observed around bus station of the existing drainage served for a long period of time. As it can be seen in the figures, installed reinforced concrete pipe were settled down and stagnant of fluid waste disposal. The reasons for failure was the backfill materials was deteriorated and the cover thickness become thin of which the application of highly repeated traffic loads may induced stress on installed reinforced concrete pipe culvert near to the surface. It resulted to damage of structure and settlement of the bedding surface. Additionally, backfill materials eroded from the pavement road was blocked the drainage outlet and disturb the flow of waste materials.

Figure 3.3photos of drainage across and along the side of road around Awitu River

From figure 3.3 at the location of manhole, there were visible defects in which from the upper stream parts Weathered pipe was observed that its backfill materials has been removed away and broken as well as cracked pipe was joining the newly constructed structure. Here boulder stone materials and earthen soil were moved to the structure that deposited on the entrance of pipe. Uncovered manhole also facilitate the silt accumulation inside the pipe. From the observation bar or only side backfill from one direction of the newly installed pipe which caused lateral pressure or stress coming from road side that starts to crack on the joints of concrete.

3.3 study Procedure

To achieve the objectives of the research, procedures have been followed. The research work was experimental and empirical analysis of different design data performing the following activities like assessment of different relevant literature review, gathering necessary data from Ethiopian Road Construction Corporation at Jimma district and Metaferia consulting office, and finally samples was collected from quarry site and field density test from where the backfill materials has been brought and site visit and observation at the installed pipe location in the Jimma town. After accomplishing these steps a series of laboratory tests has been conducted that include grain size analysis (sieve, hydrometer), compaction (MDD, OMC), Atterberg limits (liquid limit, plastic limit). After thoroughly accomplishing all the above steps of activities, a conclusion and recommendation has been drawn.

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1Field Test

From the observation of site installation condition of pipe both along and across the pavement road were taken photos from newly constructed road around Awitu River and other from bus station which are deteriorated. Samples of backfill materials were collected from three quarry sites at two locations. For each sites, a total of six samples were taken for laboratory test while field density for each locations were accomplished using sand cone replacement method.

Finally the field density and moisture content for each site has been calculated as shown in Table 4.9.

Table4. 1Results of sand cone replacement test

Serial no.	Sample location	Moisture content (%)	$\gamma b/\gamma d$,(g/cc)
1	SL1	8.62	1.56
2	SL2	8.64	1.58
3	JL1	14.51	1.02
4	JL2	14.25	1.16
5	ML1	13.81	1.23
6	ML2	13.78	1.31

4.2 Laboratory test

4.2.1 Compaction test

4.2.1.1 General principles

Compaction in general is the densification of soil by removal of air, which requires mechanical effort. The degree of compaction of soil is measured in terms of its dry unit weight. The soil particles rearrange over each other and move in to a densely packed position. The dry unit weight after compaction first increases as the moisture content increases. When the moisture content is gradually increased and same compaction effort is used for compaction, the weight of the soil solids in a unit volume gradually increases. Beyond a certain moisture content, any increase in the moisture content tends to reduce the dry unit weight. This is because the water takes up the spaces that would have been occupied by the solid particles. The laboratory test generally used to obtain the maximum dry unit weight of compaction and the optimum moisture content is known by proctor compaction test [5].

4.2.1.2 Laboratory procedure of compaction test

The procedure for the standard proctor test has followed using ASTM test designation D – 698(ASTM, 2001). The soil was compacted in a mold that has a volume of 944cm3, and having a diameter of 101.6mm. During laboratory test the mold was attached to a base plate at the bottom and to an extension at the top. The soil has been mixed with the increment of 2% water in each test and compacted with three equal layers by a hammer that delivers 25 blows to each layer 2.5kg of hammer has dropped from a height of 30.5cm as shown in figure 4.1. For each test the moist unit weight of compaction has been calculated

Figure 4.1.Photograph showing compaction test in laboratory

Using the data obtained from compaction test, compaction curves were plotted in figure 4.2 - 4.6to obtain the maximum dry unit weight and the optimum moisture content for the soil which was the basis for determining the percent compaction and water content needed to achieve the required engineering properties and then determining maximum dry unit weight and optimum moisture content as shown in Table 4.10. These data were used for evaluating the height of back fill materials over the pipe culvert.

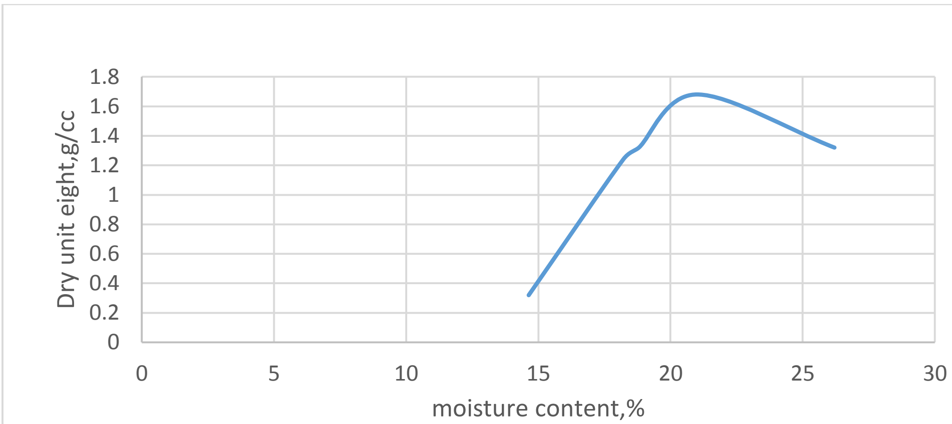


Figure 4.2.Compaction curve of Seka site at location1

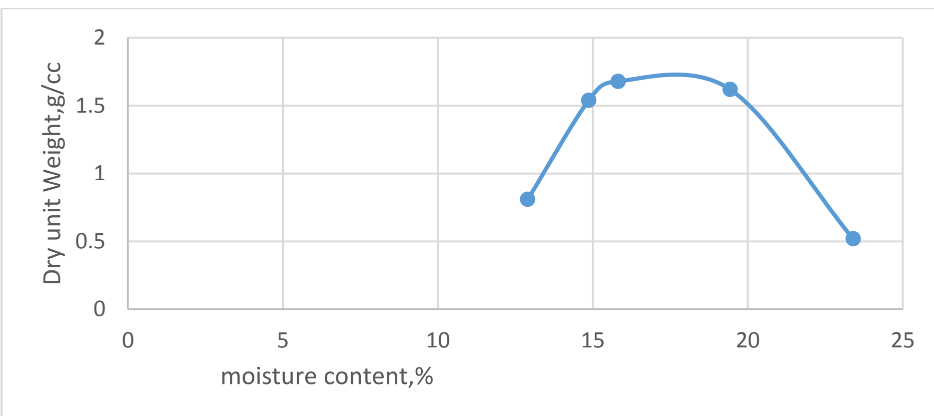


Figure 4.3Compaction curve of Seka site at location 2

From the observation of compaction curve of Seka site at both locations, Figure 4.2 shows that there is slightly decrease after reaching the maximum point while Figure 4.3, bends down abruptly and both compaction curve stands for gravely sand according to ASTM test designation D-698[2].

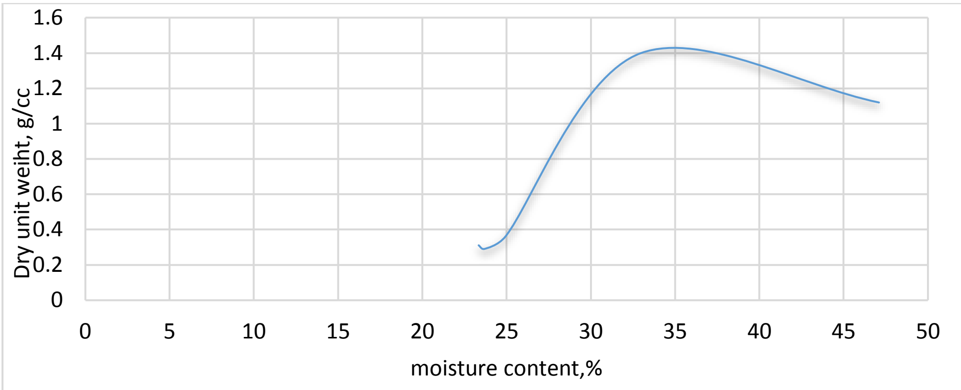


Figure 4.4Compaction curve of Merewa site at location 1

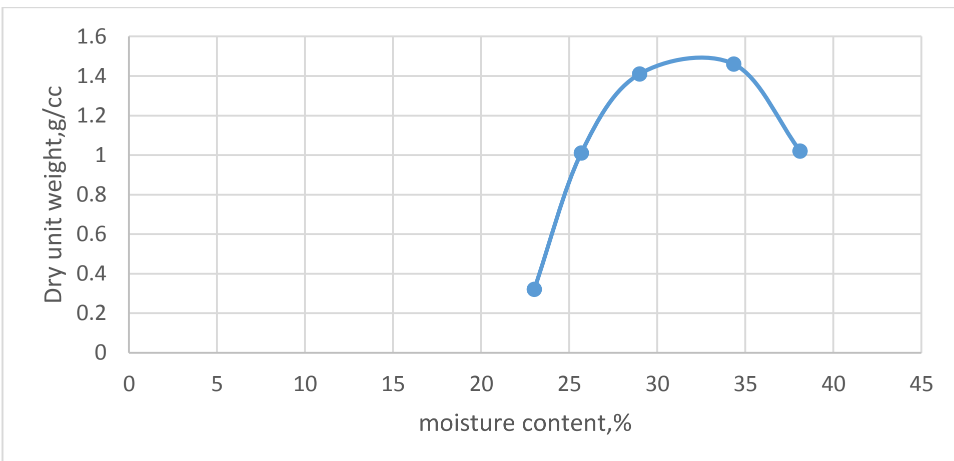


Figure 4.5Compaction curve of Merewa site at location 2

In the above figures4.4 and 4.5, the curves are almost similar but the first curve is smoothly decreases beyond the maximum point as compared with the second curve which is abruptly decrease. These curves are the property of sandy silt according to ASTM test designation D-698[5].

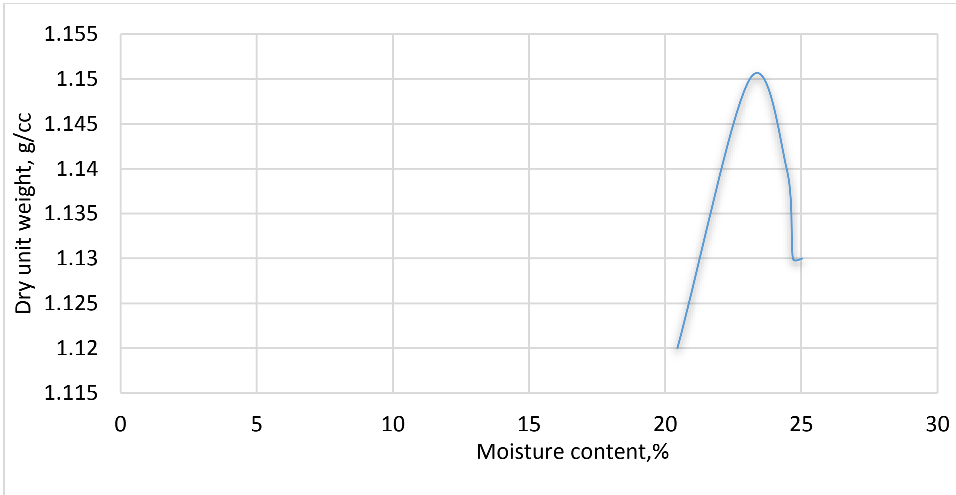


Figure 4.6Compaction curve of Jiren site at location 1

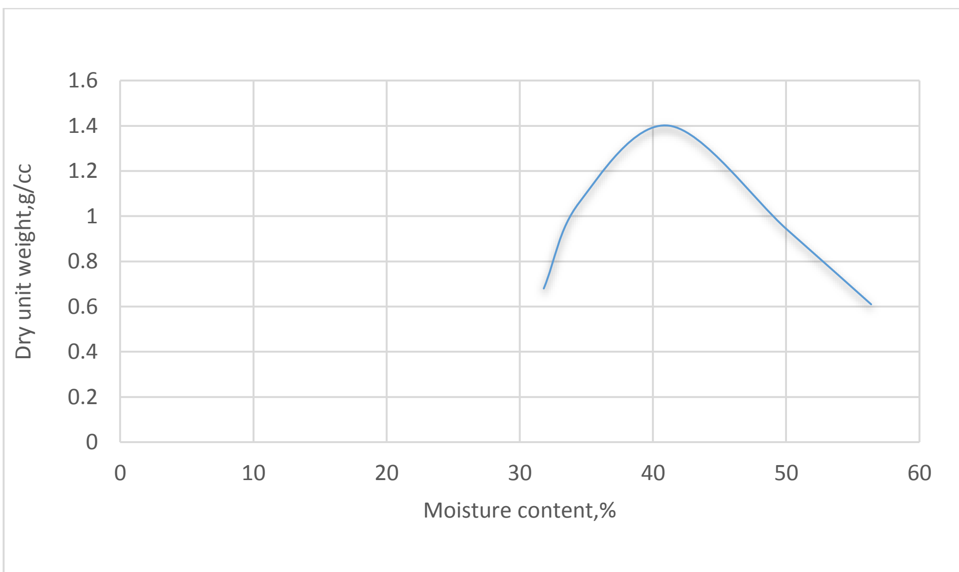


Figure 4.7Compaction curve of Jiren site at location 2

Comparison of the compaction curves of Jiren site at location 1 and location 2, the first curve showed sharp curve at maximum dry unit weight, but the second curve was smooth curve at the peak point. These two curves represent property of silt clay backfill materials according to ASTM test designation D-698 [5].

Table 4.9Compaction test result

Serial no	sample location	Field moisture content	F.D.*D* (g/cc)	MDD(g/cc)	OMC (%)	Relative compaction R.C=F.D.D/MDD*100 (%)
1	SL1	8.62	1.56	1.68	21.23	92.86
2	SL2	8.64	1.58	1.68	15.82	94.05
	Averag e	8.63	1.53	1.68	18.53	93.46
3	JL1	14.51	1.02	1.15	23.11	88.7
4	JL2	14.25	1.16	1.4	41.21	82.86
	Averag e	14.38	1.09	1.28	32.16	85.78
5	ML1	13.81	1.23	1.32	32.97	93.18
6	ML2	13.78	1.31	1.46	35.23	89.73
	Averag e	13.8	1.27	1.39	34.1	91.46

Seka site has maximum average relative compaction of 93.46 percent while Jiren site has a lowest value of 85.78 percent. From this, it is clearly indicated that Seka site is the most preferable backfill materials of all the other sites for thickness cover for reinforced concrete pipe culvert.

Table 4.10. Comparison of laboratory tests of proctor compaction density with specification

Installation type	category of back fill materials	description of category	Percent(%) proctor compaction density			
			site	Laboratory result	AASHTO specification	remarks
1	I	Gravely sand	Seka	93.46	95 -100	Out of limit
	II	Sandy silt	Merewa	91.46	95-100	“
	III	Silt clay	Jiren	85.78	100	“
2	I	Gravely sand	Seka	93.46	90-100	Within limit
	II	Sandy silt	Merewa	91.46	90-100	within limit
	III	Silt clay	Jiren	85.78	95-100	“
3	I	Gravely sand	Seka	93.46	85-100	Within limit
	II	Sandy silt	Merewa	91.46	90-100	“
	III	Silt clay	Jiren	85.78	95-100	Out of limit
4	I	Gravely sand	Seka	93.46	No compaction required	Within limit
	II	Sandy silt	Merewa	91.46	No compaction required	Within limit
	III	Silt clay	Jiren	85.78	85	Within limit

From table 4.11 above backfill materials of Seka site having maximum percentage of proctor compaction density of 93.46fulfills the criteria of installation Type II, Type III Type IV and within the limits of AASHTO specification, but out of limit for installation Type I.

Backfill materials of Merewa site has medium percentage of proctor compaction density of 91.46 fulfills the criteria for installation Type III and Type IV only. Similarly, backfill materials of Jiren site has lowest percentage of proctor compaction density of 85.78 fulfills the criteria for installation Type IV of Category III back fill materials only based on standard specification.

4.2.2 Index properties

4.2.2.1 General concepts

The standard test used for performing the test was ASTM D-422 to determine the index properties of the materials. The purpose of conducting laboratory tests of index properties was to determine their physical properties mainly for identification and classification purposes of grain size, Atterberg limits.

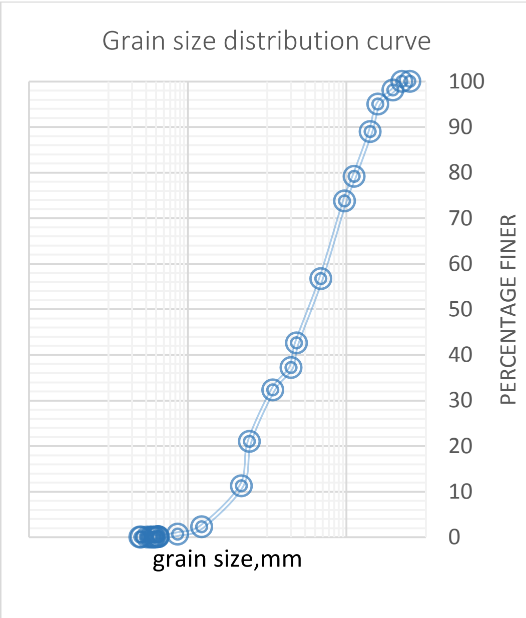
4.2.2.2 Gradation test

Grain size distribution is the basic soil property which affect its Engineering properties considerably and used in most soil classification system. Mechanical sieve analysis has used to determine the grain size distribution of coarse grained soils such as sand, and for fine grained soils hydrometer analysis is used for determining the distribution of grain size [21].

Particle size distribution curve as shown in Figure 4.8 consists of three different types of curves in which poorly graded soil is a soil with most of the soil grains are of the same size; well graded soil is a soil with a particle size are distributed over a wide area range, and gap grade soil consists of two or more uniformly graded fractions. There are two useful indicator, C_u and C_c which are obtained from the grain size distribution curve; C_u is uniformity of coefficient and it is defined as $C_u = d_{60}/d_{10}$, C_c is the coefficient of gradation, which is defined as $C_c = (d_{30})2/ (d_{10}*d_{60})$, where d_{10}, d_{30}, and d_{60} are the grain diameter corresponding respectively to 10%, 30%, and 60% passing or percent finer on the gradation curve [5, 6& 4].

4.2.2.3 Test results

The test method followed were ASTM D-422 and the grain size analysis were determined through performing sieve analysis test and the results were presented in Tables and Figures. Figures4.9 up to 4.14 show grain size distribution curve of combined coarse and fine grained materials of the test result for the proposed quarry sites that are used for backfill materials over the reinforced concrete pipe. From the result obtained and comparing with the Unified soil classification system Seka site of both at location one and two are well graded sand with gravel, but each of Jiren and Merewa site are poorly graded sand with gavel.



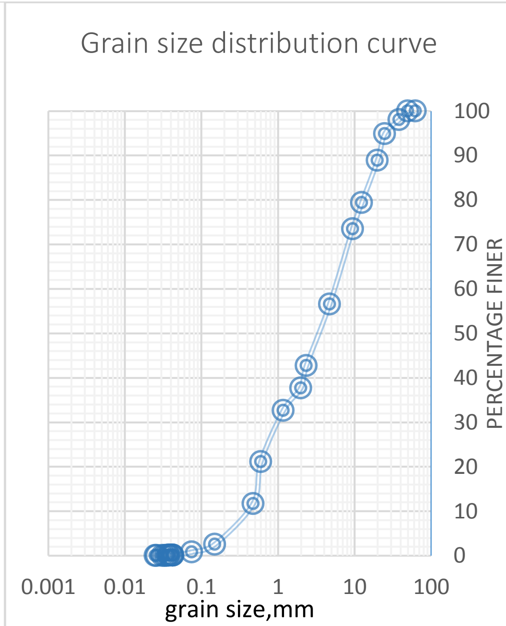
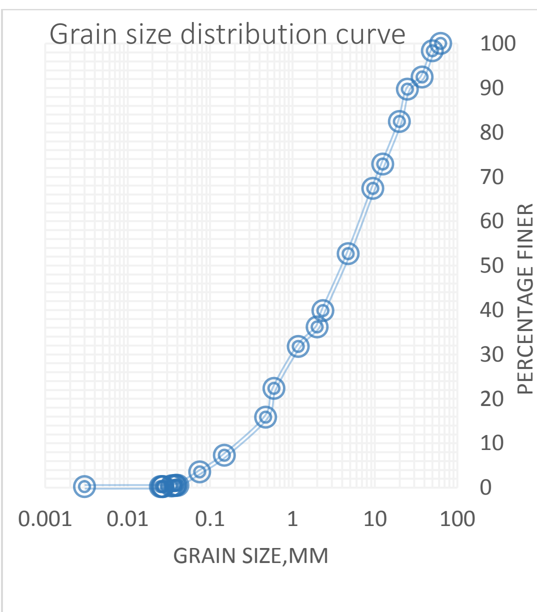


Figure 4.8Grain size distribution of Jiren site at location 1Figure 4.9Grain size distribution of Jiren site at location 2



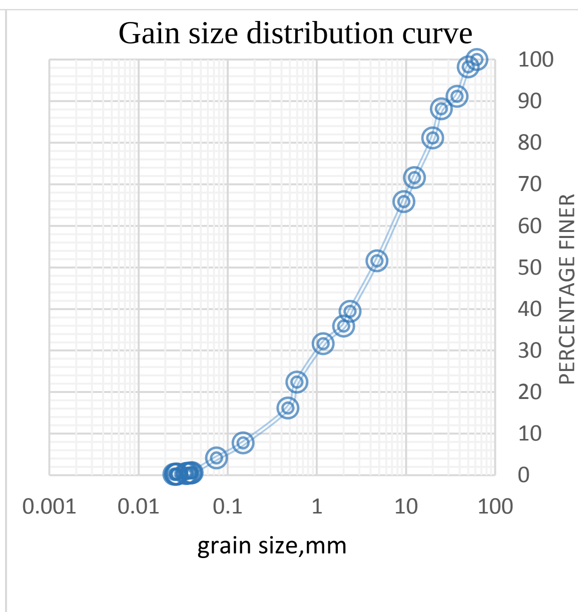
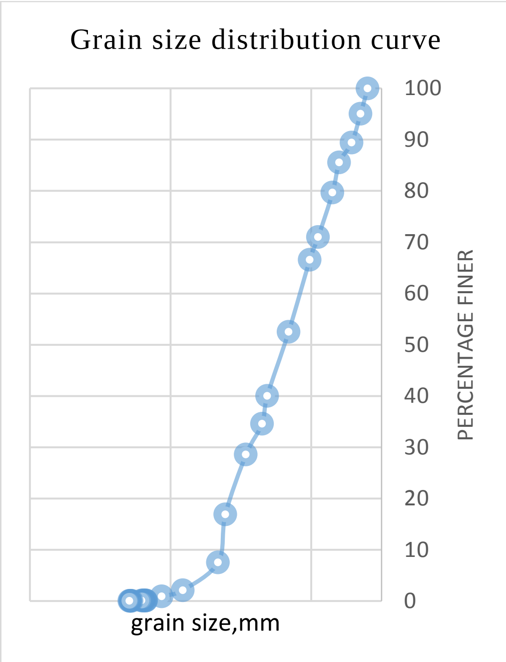


Figure 4.10Grain size distribution of Seka site at location 1Figure 4.11Grain size distribution of Seka site at location 2



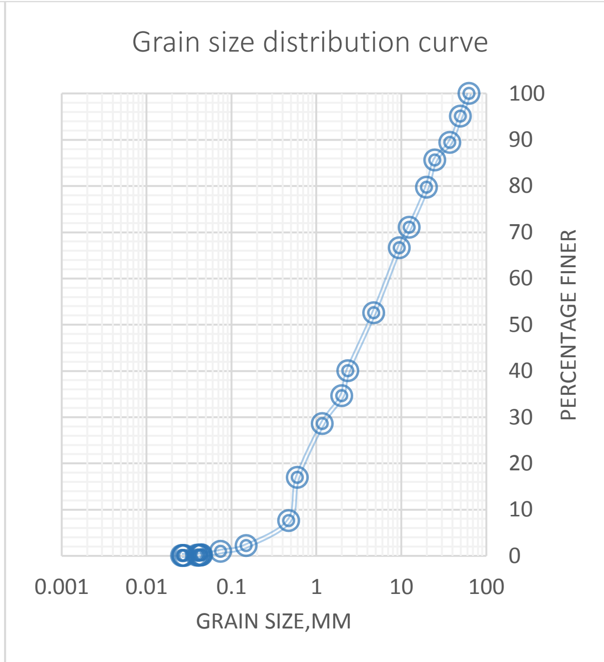


Figure 4.12Grain size distribution of Merewa site at loction1Figure 4.13Grain size distribution of Merewa site at 2 location 2

Table 4.11Classification of coarse grained materials (using USCS)

Item No	Sample location	D10	D30	D60	CU	Cc	Soil type	soil group Name
1	SL1	0.25	1.35	7.13	28.52	1.02	SW	well graded sand with gravel,
2	SL2	0.22	1.32	7.10	32.27	1.12	SW	well graded sand with gravel,
3	JL1	0.43	1.06	5.47	12.72	0.6	SP	-Poorly graded sand with gravel
4	JL2	0.41	1.05	5.71	13.93	0.47	SP	Poorly graded sand with gravel
5	ML1	0.50	1.37	7.02	14.04	0.53	SP	Poorly graded sand with gravel
6	ML2	0.51	1.37	7.28	14.27	0.51	SP	Poorly graded sand with gravel

4.2.3 Specific gravity

The main purpose of determining the specific gravity of soil is used to input such value in calculating the hydrometer test analysis. This test method covers the determination of the specific gravity of soils that pass the 2.00mm (No.10) sieve. The specific gravity of soil samples under evaluation was determined using ASTM D854-92standard, and the results obtained are tabulated in Table (4.13).

Table 4.12Specific gravity test result

Item No.	sample location	Specific gravity (GS)
1	SL1	2.65
2	SL2	2.67
3	ML1	2.68
4	ML2	2.69
5	JL1	2.7
6	JL2	2.72

Where G_s stands for specific Gravity and it is given by density of particle (ρ_s) divided by density of water (ρw)or simply explained as$G_s = \frac{\rho_s}{\rho_w}$.

4.2.4 Atterberg limit

Atterberg limits are regarded as useful indices for determining the characteristics of most clay. This is true because parameters depend on the amount of water a soil tries to imbibe. A typical soil mass have three constituents: soil grains air and water. In soils consisting largely of fine grains, the amount of water present in the void has a pronounced effect on the soil properties. When a clay soil is mixed with an excessive amount of water it may flow like a semiliquid. If a soil is gradually dried, it will behave like plastic, semisolid, or solid material depending on its moisture content. The moisture content, in percent at which the soil changes from a liquid to plastic state is defined as the liquid limit (LL). Similarly, the moisture content, in percent at which the soil changes from plastic to semisolid is defined as plastic limit (PL).Fine soils were getting enough moisture even without keeping wet for longer duration. Hence one can carry out Atterberg limit tests without keeping soil specimens wet for 24hrs for moisture content equilibration. But for this thesis work all Atterberg limit tests were carried out on soil specimens kept wet for 24 Hrs. These tests are performed on the basis of air dried sample passing the 0.425 mm sieve size [5].

Table *4.13*Summarized values of liquid limit and plastic limit results

Sample location	liquid limit,%	plastic,%
Seka site at location 1	28	3
Seka site at location 2	30.2	5.2
Average	29.1	4.1
Jiren site at location 1	65	17.2
Jiren site at location 2	60.5	16.5
Average	62.75	16.85
Merewa site at location 1	51.3	12.8
Merewa site at location 2	43.9	10.9
Average	47.6	11.85

Table 4.14Atterberg limit test result for fine grain material

Sample location	Liquid limit, %	plastic limit, %	plastic index, %	Group classification	Plasticity classification		soil type(AASHTO)
					AASHTO specification	Plasticity	
SL1	28	25	3	A-2-4	< 7	Low	clayey gravel and sand
SL2	30	25	5	A-2-4	< 7	Low	clayey gravel and sand
JL1	65	47.8	17.2	A-7-5	> 17	High	clayey soil
JL2	60.5	44	16.5	A-7-5	7-17	medium	clayey soil
ML1	51.3	38.5	12.8	A-7-5	7 - 17	medium	silty/clayey gravel and sand
ML2	43.9	33	10.9	A-2-7	7 - 17	medium	silty/clayey gravel and sand

From Table (4.15) above for different sample location there was different results of plasticity index and comparing these respective values with the given specification, the degree of plasticity has been determined and finally soil type of fine grain material has been identified. Hence from this, Jiren site has the highest plasticity index and its soil type is clayey soil which is not appropriate as a backfill materials.

4.3Discussion of laboratory test results

4.3.1 Compaction test

From the compaction test results tabulated in Table 4.10, it has been clearly observed that Seka site has a higher maximum dry density than Jiren and Merewa sites and its optimum moisture content is lowest in comparison with the others. However, it has an average largest percentage proctor density. The Maximum dry density of Merewa was medium, but its average percentage proctor density was 91.46% somehow nearest to Seka site that is having average percentage of proctor density of 93.46%. The Jiren quarry site has a maximum dry density of of 1.28 with its optimum moisture content of 32.16% and percentage proctor density of 85.78% which represent lowest value compared to the sites. This indicates that the compactive effort of backfill materials from Seka site is very well than Jiren quarry site.

From the Table (4.11) all sites did not fulfill the specification under installation type I, through which their percentage proctor compaction are less than the specified limit. The back fill materials from the three sites have observed that it did not meet the criteria stated in the standard specification. While Considering the two quarry sites (Seka and Merewa), these sites has fulfilled the limit for both Installation type II as well as type III ,but the backfill materials from Jiren borrow pit could not be use for type II and type III installation. Finally, for installation type IV it requires no compaction under backfill materials of Seka and Merewa sites, but for Jiren site the category of the backfill material is silty clay. This case needs to have a minimum of 85% proctor compaction based on standard specification.

4.3.2 Gradation test

The gradation test result showed Seka site was classified as well-graded sand with gravel and this indicates that it is appropriate to use for backfilling materials over the surface of reinforced concrete pipe. But Jiren and Merewa sites, were classified under poorly graded sand with gravel. From this result, the two sites need mixing of fine and coarse materials in order to obtain well-graded back fill materials for drainage structure construction.

4.3.3 Atterberg limit test

From Table (4.15) different sample locations were undertaken and it was observed that there were different results of plasticity index. These values are compared with the given standard specification. The degree of plasticity has been determined and finally the soil type of fine grain material has been identified. Hence from this fine materials of Seka quarry site has low plasticity while Jiren quarry site has highly plasticity and Merewa quarry site has medium plasticity. Soil type is categorized as silty –clayey gravel sand for the case of Seka site, clayey soil for Jiren site and clayey soil at location (1), clayey gravel and sand at location (2) for Merewa site. Therefore using backfill materials from Seka site is more preferable. It was observed that Jiren site is less appropriate as a back fill materials.

4.3.5 Three edge bearing analysis using pipe Pac software

Three edge bearing is important since it allows the user to “customize” site conditions in which concrete pipe is to be placed. Input parameters including pipe shape could be easily modified in order that numerous loading and installation scenarios was modeled in a matter of time.

Another useful feature of three edge bearing analysis is that variable bedding factor and variable arching factor could be specified and thus a variable bedding factors were specified for bedding type B and C. The variable arching factors were specified for bedding type I, II, III and IV in which these factors were considered the moment induced in a section of pipe after placement and backfilling, is less severe than the moment induced in a pipe section by the standard three edge bearing test. In considering this information, pipe of lesser strength could be exploited. It is also effective in time saving by selecting more than one bedding type in a single analysis.

Finally, three edge bearing analysis provides clear and concise tabular output where D-load values to produce a 0.3mm width of crack over the surface of pipe were listed and in multiple bedding type selection summary table. It has been viewed comparing the D-loads which shows each incremental pipe depth. Therefore, three edge bearing provides a way to calculate all bedding types and compare each results.

In running the software the following dialogue box has showed the procedural analysis of three edge bearing test of reinforced concrete pipe for the three sites.

From the three sites, Seka site was selected for explanation as a sample and Jiren and Merewa sites were attached at the end. In the dialogue box starting from figure (4.21) up to figure (4.25) were presented as follows:

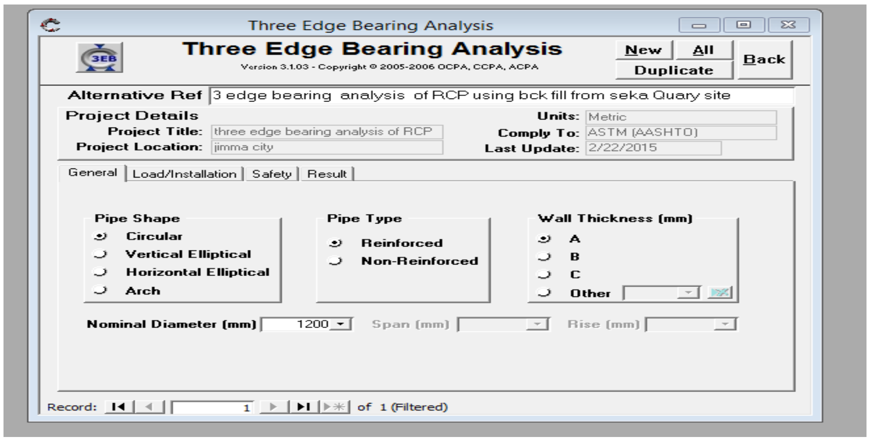


Figure 4.14pipe shape, type and wall thickness

Where A stands for wall thickness type and it is given by,$h = \frac{D_i}{12}$ h refers wall thickness and D_i is inside diameter of Circular pipe of reinforced concrete pipe with the nominal diameter of 1200mm has selected for the three edge bearing analysis in order to get out required D- load for 0.3mm crack width which were selected as a reference that the machine simply detect and it can be maintained.

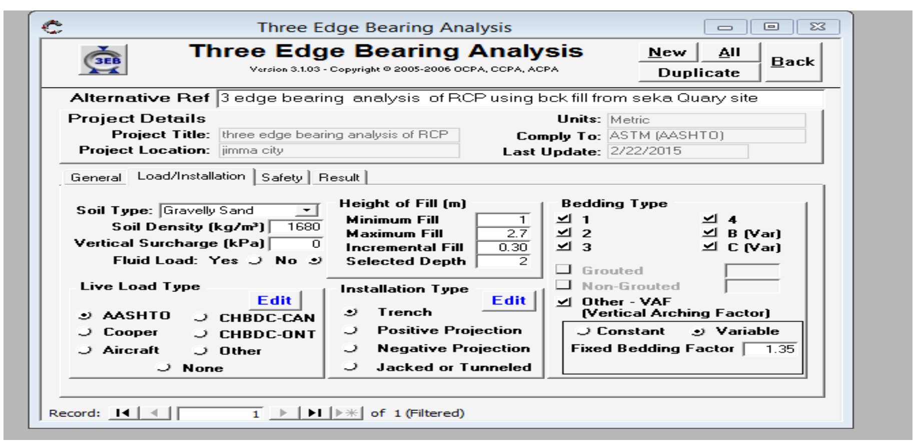


Figure 4.15soil property and fill height

The soil type obtained from laboratory result for Seka site was gravely sand with density of 1680kg/m3 and height of fill actually used on the installed condition were ranges from 0.3m to 1.5m at an incremental fill of 0.3m. Selected depth for evaluating the D-load requirements was 2m and installation type was trench excavation, live load type used was AASHTO standards and all bedding type has been checked at a time and for each bedding type the output was displayed sequentially.

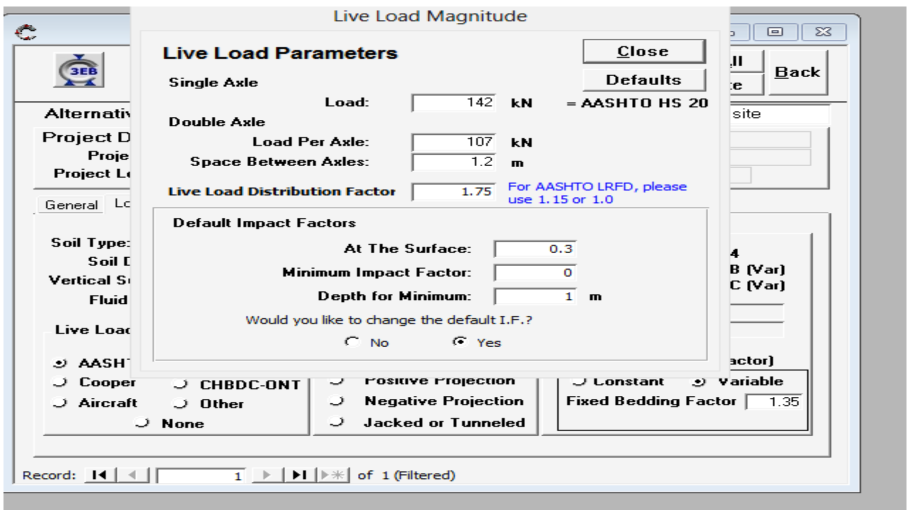


Figure 4.16Live load parameters

The approach used in pipe Pac software was stated as loads were applied in a direction of travel perpendicular to the axis of a pipe and in a direction of travel parallel to the axis of pipe and the worst case condition was used for analysis.

The live load type used for analysis was AASHTO HS 20 which stated for both single axle and double axle with the value of load per axle 142kN and 107kN respectively at live load distribution factor of 1.75 and 1.2m spacing between axles has been used over the range of fill height specified. This was used to know the live load distribution at each incremental depth of 0.3m along the vertical section of trench backfill.

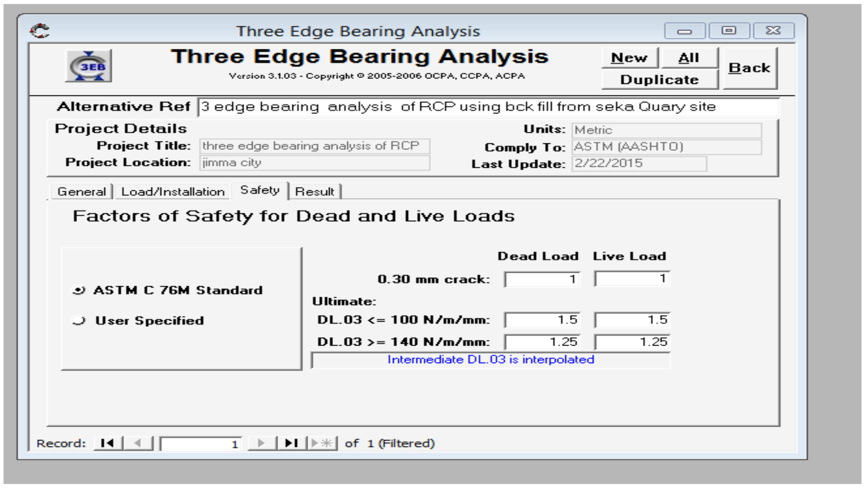


Figure 4.17Factor of safety for dead and live loads

Finally factor of safety (f.s) for dead and live load was utilized as per ASTM C 76M standard. The total earth load and live load on a buried reinforced concrete was computed and multiplied by a factor of safety in order to determine the pipe supporting strength. Factor of safety is the relationship between ultimate strength D-load (D_{ult}) and the 0.3mm crack D-load ($D_{0.3}$). Ultimate D-load less than or equal to 100N/m/mm the same factor of safety of the value 1.5 for both dead load and live load was used, but for ultimate D-load greater than or equal to 140N/m/mm factor of safety of 1.25 was used based on Canadian Standard specification.

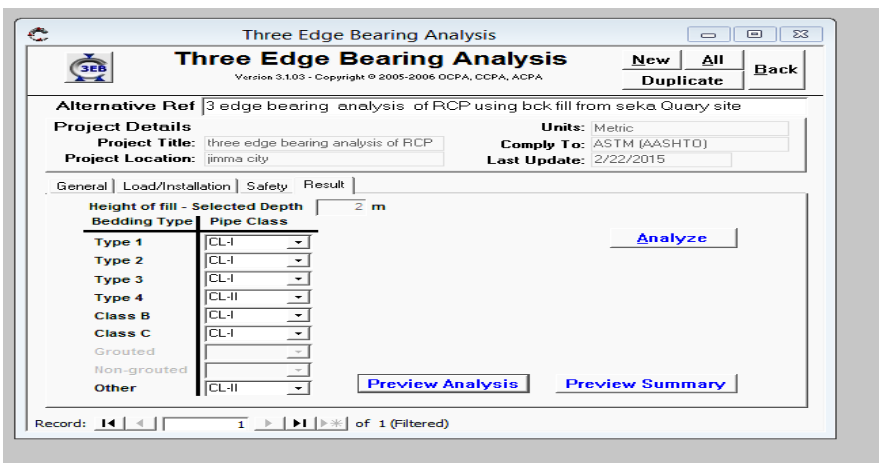


Figure 4.18Bedding type and pipe class

From the dialogue box, height of fill for selected depth for analysis was 2m and each bedding type has been analyzed for different pipe classes which decides required D-load that creates

0.3mm crack over the surface of the reinforced concrete pipe and for each of them results were displayed on the consecutive tables.

D- Load requirements for a 1200mm diameter of reinforced concrete circular pipe results of analysis obtained from three edge bearing for each bedding type were shown in the next tables. The selected depth for analysis was 2m which is nearest to 2.2m and discussion for each term were explained below.

Table 4.15Results of analysis for bedding others

Pipe depth	Earth Load			Live load(kN/m)	Total Load(kN/m)	Bedding factor		Required D-load 0.3mm(N/m/mm)	Ultimate load with 1.5 factor of safety
	Arching factor	>Trans	Load(kN/m)			DL	LL		
1	1.13	Y	26	19	45	1.35	1.7	28(CL-I)	42
1.3	1.17	Y	35	14	49	1.35	1.7	30(CL-I)	45
1.6	1.21	Y	45	11	56	1.35	1.7	35(CL-I)	52.5
1.9	1.26	Y	55	9	64	1.35	1.7	40(CL-I)	60
2.2	1.31	Y	66	8	74	1.35	1.69	46(CL-II)	69
2.5	1.35	Y	78	6	85	1.35	1.68	52(CL-III)	78
2.7	1.36	Y	85	6	91	1.35	1.68	56(CL-III)	84

The results of analysis was displayed for bedding type others and this satisfy pipe strength of class I, class II and class III. Arching factor refers the ratio of the design soil load on the pipe to the load from the prism of soil immediately above the pipe. Trans which Shows transition width at which the trench width is no more give support for pipe from side friction and a symbol of ‘Y’ was shown when the trench width exceeds the width at which frictional forces reduce the soil load on the pipe that in bedding type others the trench width was greater than transition width. Hence, there was no frictional force which give support from the side of the pipe.

In all the cases required D load for 0.3mm crack at each pipe depth were within the specification and the ultimate load is nearest to total load, therefore the pipe was almost safe.

Table 4.16Results of analysis for bedding type I.

Pipe depth	Earth Load			Live load(kN/m)	Total Load(kN/m)	Bedding factor		Required D-load 0.3mm (N/m/mm)	Ultimate load with 1.5 factor of safety
	Arching factor	>Trans	Load(kN/m)			DL	LL		
1	1.35	Y	36	19	55	3.94	2.2	15(CL-I)	22.5
1.3	1.35	Y	45	14	59	3.94	2.2	15(CL-I)	22.5
1.6	1.35	Y	54	11	65	3.94	2.2	16(CL-I)	24
1.9	1.35	Y	64	9	73	3.94	2.2	17(CL-I)	25.5
2.2	1.35	Y	73	8	81	3.94	2.2	18(CL-I)	27
2.5	1.35	Y	83	6	89	3.94	2.2	20(CL-I)	30
2.7	1.35	Y	89	6	95	3.94	2.2	21(CL-I)	31.5

At 0.3m incremental depth the arching factor were the same and trench width has no more given support for pipe from side friction. The dead load was increasing along the depth while live load distribution was decreasing and there was no surcharge load, so the total load was the summation of dead load and live load only. Bedding factor were presented for both dead load and live load; this bedding type satisfy the pipe strength of class I only. The results of D –Load needed for a 1200mm of circular pipe at selected depth 2m and closest pipe depth was 2.2m of reinforced pipe Classes for 0.3mm crack were less than the standard specification as per ASTM C76M (N/m/mm) which stated the required D load limits for each pipe classes and it were given as: CL I<=40; CL II<= 50; CL III<= 65; CL IV<= 100; CL V<= 140.In bedding type I required D- load at each pipe depth were within the limits of specification but total load were much greater than the ultimate load and hence the pipe would starts to fail.

Table 4.17Results of analysis for bedding type 2

Pipe depth	Earth Load			Live load(kN/m)	Total Load(kN/m)	Bedding factor		Required D- load 0.3mm(N/m/mm)	Ultimate load with 1.5 factor of safety
	Arching factor	>Trans	Load(kN/m)			DL	LL		
1	1.4	Y	37	19	56	2.87	2.2	18(CL-I)	27
1.3	1.4	Y	47	14	61	2.87	2.2	19(CL-I)	28.5
1.6	1.4	Y	57	11	68	2.87	2.2	21(CL-I)	31.5
1.9	1.4	Y	66	9	75	2.87	2.2	23(CL-I)	34.5
2.2	1.4	Y	76	8	84	2.87	2.2	25(CL-I)	37.5
2.5	1.4	Y	86	6	92	2.87	2.2	27(CL-I)	40.5
2.7	1.4	Y	92	6	98	2.87	2.2	29(CL-I)	43.5

The arching factor were fixed values at each incremental depth of 0.3m and trench width has no more given support for pipe from side friction. The dead load distribution increases while live load were decreased with zero surcharge load down the depth. The required D- load were within the range of standard specification for bedding type II in which it satisfy pipe strength of class I only. But the ultimate strength load was much less than the total load in which the pipe continue to crack more than 0.3mm width and it would fail

Table*4.18*Results of analysis for bedding type III

pipe Depth	Earth Load			Live load(kN/m)	Total LoadKN/m)	Bedding factor		Required D- Load 0.3mm (N/m/mm)	Ultimate load with 1.5 factor of safety
	Arching factor	>Trans	Load(KN/m)			DL	LL		
1	1.4	Y	37	19	56	2.27	2.2	21(CL-I)	31.5
1.3	1.4	Y	47	14	61	2.27	2.2	22(CL-I)	33
1.6	1.4	Y	57	11	68	2.27	2.2	25(CL-I)	37.5
1.9	1.4	Y	66	9	75	2.27	2.2	23(CL-I)	34.5
2.2	1.4	Y	76	8	84	2.27	2.2	25(CL-I)	37.5
2.5	1.4	Y	86	6	92	2.27	2.2	27(CL-I)	40.5
2.7	1.4	Y	92	6	98	2.27	2.2	29(CL-I)	43.5

The results of analysis for Bedding type III was similar to bedding type II, only it differ in bedding factor's values and the required D-load, which was greater than the bedding type II, still it was within the limits of standard specification. The ultimate strength load was less than the total load so that the pipe would starts to fail through time.

Table *4.19*.Results of analysis for bedding type IV

pipe Depth	Earth Load			Live load(kN/m)	Total LoadKN/m)	Bedding factor		Required D-Load 0.3mm (N/m/mm)	Ultimate load with 1.5 factor of safety
	Arching factor	>Trans	Load(KN/m)			DL	LL		
1	1.45	Y	37	19	57	1.7	1.7	28(CL-I)	42
1.3	1.45	Y	48	14	62	1.7	1.7	31(CL-I)	46.5
1.6	1.45	Y	59	11	70	1.7	1.7	34(CL-I)	51
1.9	1.45	Y	69	9	78	1.7	1.7	38(CL-I)	57
2.2	1.45	N	79	8	86	1.69	1.69	43(CL-II)	64.5
2.5	1.43	N	87	6	94	1.68	1.68	46(CL-II)	69
2.7	1.41	N	93	6	98	1.68	1.68	49(CL-II)	73.5

In type IV bedding there was variable arching factors down the backfill depth and the trench width has no more given side friction support up to a depth of 1.9m but, starting from 2.2m to 2.7m trench width has provided side friction support or the transition width has not been exceeded. In the similar manner with the above bedding types, the dead load and live load distribution increases and decreases respectively while, bedding factor's values were vary along the incremental depth. The required D- load were within the range of standard specification for this bedding type in which it satisfy pipe strength of class I and class II only.

Table *4.20*Results of analysis for bedding type B

pipe Depth	Earth Load			Live load(kN/m)	Total LoadKN/m)	Bedding factor	Required D-Load 0.3mm (N/m/mm)	Ultimate load with 1.5 factor of safety
	Arching factor	>Trans	Load(Kn/m)			DL		
1	1.13	Y	26	19	45	1.7	20(CL-I)	30
1.3	1.17	Y	35	14	49	1.7	22(CL-I)	33
1.6	1.21	Y	45	11	56	1.7	25(CL-I)	37.5
1.9	1.26	Y	55	9	64	1.7	29(CL-I)	43.5
2.2	1.31	Y	66	8	74	1.69	34(CL-I)	51
2.5	1.35	Y	78	6	85	1.68	38(CL-I)	57
2.7	1.36	Y	85	6	91	1.68	41(CL-II)	61.5

In type B bedding there was variable arching factor that it increased as depth of trench increase. The trench width has no more given side friction support trough out the depth. There was only dead load bedding factor, and the required D- load for 0.3mm crack was with in the standard specification in which it satisfy pipe strength class I and class II only but the ultimate strength load was less than the total load so that the pipe would starts to fail gradually.

Table 4.21Results of analysis for bedding type C

pipe Depth	Earth Load			Live load(kN/m)	Total LoadKN/m)	Bedding factor	Required D-Load 0.3mm (N/m/mm)	Ultimate load with 1.5 factor of safety
	Arching factor	>Trans	Load(Kn/m)			DL		
1	1.13	Y	26	19	45	1.7	20(CL-I)	30
1.3	1.17	Y	35	14	49	1.7	22(CL-I)	33
1.6	1.21	Y	45	11	56	1.7	25(CL-I)	37.5
1.9	1.26	Y	55	9	64	1.7	29(CL-I)	43.5
2.2	1.31	Y	66	8	74	1.69	34(CL-I)	51
2.5	1.35	Y	78	6	85	1.68	38(CL-I)	57
2.7	1.36	Y	85	6	91	1.68	41(CL-II)	61.5

In bedding type C the arching factor were variable and it increased along the depth of trench. The trench width has no more given side friction support throughout the backfill of depth. The dead load and live load distribution increases and decreases respectively. There was Bedding factor for dead load and it decreases down, this means the support given by the surrounding soil was become decreased. The required D- load were within the range of standard specification for this bedding type in which it satisfy pipe strength of class I and class II only.

Generally as D-load for 0.3mm crack was the required load to support in the three edge bearing test with a crack equal to or less than 0.3mm which equates to the maximum stress induced on the pipe in the installed condition for each bedding type.

The total load increases as depth of pipe increases and in each of bedding types required D-load was less than total load and it increased along the backfill depth. From this it has been observed that 0.3mm crack was occurred easily. As the depth of pipe moved down ward required D-load was further increase. Bedding factor for D.L in bedding Type I was greater than Type II, Type III and Type IV, but bedding factor for live load of Type I, II and III were the same and it keeps decreasing for type IV along the depth.

4.2.6 Evaluation of back fill height

Table 4.22Comparison of backfill height with the specification

Installation Type	Pipe Diameter, m. mm	AASHTO standard specification					Results obtained from actual Design of the construction company			
		Maximum Height of Fill (m)			Minimum Allowable Cover Height (m)		Maximum Height of Fill (m)			Minimum Allowable Cover Height (m)
		AASHTO 170M					AASHTO 170M			
		Class III	Class IV	Class V	HS-20 Vehicle Loading	Construction Vehicle Loading.	Class III	Class IV	Class V	HS-20Vehicle Loading
Type I	300-900	8.4	12.4	12.4	0.3	0.9	-	-	-	- -
	105-165	8	12	18	0.3	0.9	-	-	-	-
	180-240	7.8	12	17.7	0.3	0.9	-	-	-	-
Type II	300-750	5.9	8.7	13	0.3	0.9	1.5			0.3
	900-2400	5.5	8.4	12.7	0.3	0.9	1.5			0.3
Type III	300-1000	4.3	6.5	10	0.3	0.9	1.5	-	-	0.3
	1200-2400	4	6.5	10	0.3	0.9	1.5	-	-	0.3
Type IV	300-525	9	4.3	6.5	0.3	0.9	1.5			0.3
	600-2400	9	4.6	7	0.3	0.9	1.5			0.3

Table (4.23) contain the necessary criteria which helps to evaluate and discusses about the backfill height. For the sake of installation and act as the property of site condition reinforced concrete pipe were classified under four installation type depending on the pipe strength and back fill material category. For the purpose of analysis for the maximum and minimum cover thickness over the top of concrete pipe, it has been taken the above data from AASHTO specification and different researches that the comparison was done.

The diameter of pipe culvert used in Jimma town runs from 750mm up to 1200mm with the maximum back fill height of 1.5m and minimum 0.3. Then the issue was comparing these actual installation with the above back fill height ranges which relates the type of installation and classes of reinforced concrete pipe culvert by respective values of maximum fill height and minimum allowable cover thickness. From the three location only Seka site full fill the criteria for type II installation which allows silty granular soils with less compaction effort, but Seka and Merewa could use as a backfill materials for type III installation which allows use of soils with less stringent compaction requirements. Finally it has been observed that all quarry sites were within the required range of height. In each of installation type mentioned above except installation type I which allows relatively high quality materials and high compaction effort with lower strength pipe class III reinforced pipe culvert was used in the study area. The maximum back fill height actually used on the site was not beyond the limits of specification and the minimum allowable cover height was almost close to the specified values and it was greater than the required cover thickness.

4.2.6.1 Boussinesq's Theory of stress analysis

Boussinesq (1885) has given the solution for the stress caused by the application of the point load at the surface of an elastic medium to find out the depth at which stress influence could be neglected and determine bedding depth under the bottom of the pipe culvert. Then find out the maximum depth from the top of pipe culvert to bottom layer of pavement using the live load data obtained from three edge bearing analysis for each site.

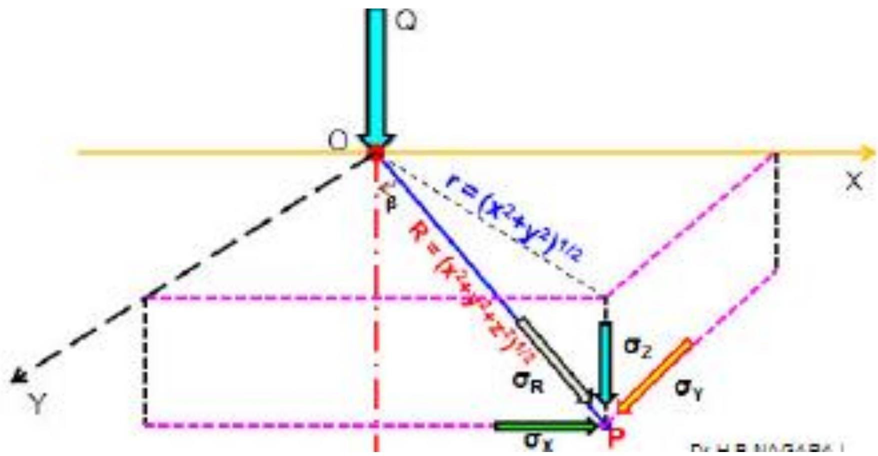


Figure 4.19 Diagrammatic presentation of Point loadapplication.

$$\delta \;=\; \frac{3Q}{2\pi}\frac{1}{Z^2}\left(1/(1\;+(\frac{r}{z})^2)\;{}^{5}\!/_{2}\right)$$

The coefficient IB is known as Boussinesq influence for the vertical stress and it is determined for the given value of:

$$IB = \frac{3}{2\pi}(1/(1\;+(\frac{r}{z})^2)^{\,5/2}$$

.Where R = polar distance between origin O and point

r = the horizontal distance between an arbitrary point P below the surface and the vertical axis through point load Q

z = The vertical depth of the point P from the surface

β = Angle which the line OP makes with the vertical

Table 4.23comparison of boussinesq's results with standard specification

S.No	site name	Installation type	Boussinesq results(m)	Maximum Required value(m)	AASHTO 170M specification(m)	Actual installation value(m)	Remarks
1	Seka	II	2.8	1.6	5.5	1.5	Within limit
		III	2.8	1.6	4	1.5	Within limit
		IV	2.8	1.6	9	1.5	Within limit
2	Merewa	II	2.6	1.4	5.5	1.5	Within limit
		III	2.6	1.4	4	1.5	Within limit
3	Jiren	IV	2.5	1.3	9	1.5	Within limit

For each installation type there is a respective standard specification of maximum depth of backfill height. In laboratory results Seka site fulfill the criteria for installation type II, type III and type IV, the backfill materials from this site could cover to the maximum depth of 1.6m, which is greater than the actual design value used by Ethiopian Road Construction Corporation but, less than AASHTO 170M standard specification. Merewa site fulfills the installation type II, and type III, this site also a maximum backfill depth of 1.4m and similarly Jiren site could use as a maximum backfill depth of 1.3m, it was less than both the actual design value and standard specification.

Therefore it was clearly showed that backfill materials from Seka site has greatest maximum depth than Merewa and Jiren site, and each of them were within the limits of standard specification of AASHTO 170M as well as actual design value used by the construction company.

CHAPTER FIVE

CONCLUSION AND RECOMMENDATION

5.1 Conclusions

From the findings of field and laboratory test results the following conclusions are drawn

- Compaction test result of the three sites, backfill materials from Seka comprises highest percentage proctor compaction density while Merewa quarry site has moderate but Jiren shows low value as compared with the other. Therefore, these results show that backfill materials from Seka site are more suitable than the two quarry sites used as backfill materials over the top of the reinforced concrete pipe culvert.
- Seka site fulfills the criteria for installation of type II, III and IV except the backfill materials of Category I which is gravelly sand and the plasticity index of the samples were within the AASHTO specification.
- Gradation test result proved that Seka site is well- graded gravelly sand and other sites have poorly graded materials. This indicates that mixing of materials is not required for proportioning the soil particle with other type of backfill materials.
- From the Boussinesq analysis of stress influence due to point load on the backfill materials of Seka site was safe at the fill height of 1.6m and comparing this value with actual design of maximum fill height it was minimum and Merewa site maximum backfill height of1.4m and it was less than the actual value of design as well as the standard specification. Similarly for Jiren backfill materials results of analysis, the value was 1.3m, this backfill depth was still lower than both standard specification and actual design value.
- The fill height used over the top of reinforced concrete pipe by the construction team was within the limits of standard specification but the observed failure during field investigation was due to deterioration of backfill on top of pipe culvert and continuous moistening of foundation soil under the bedding of pipe which resulted

to reduction of bearing capacity of soil and its effect is settlement of bedding surface under the pipe. Therefore the distribution of load over the pipe was observed not uniform which causes crack of concrete pipe.

- The ultimate load specified in ASTMC76 or the maximum three edge bearing test load supported by a pipe with value of 42 N/m/mm is less than total applied load of 62N/m/mm, for bedding type B. therefore the crack was formed over the surface of pipe which leads to crack and through time deformation of pipe will be occurred that resulted in deterioration of pavement structure.

5.2 Recommendation

From the laboratory test it was showed that backfill materials of coarse grain of two sites were poorly graded and also their fine materials have medium to high plasticity index. For this purpose further research need to be carried out on blending the soil material with non-plastic soils.

- Cost benefits analysis should be made in the installation of concrete pipe in comparison of soil pipe interactions approach, indirect design method which classify installation of pipes in respective of their strength classes with the standard direct design,
- Backfill materials from different category of the three site (Seka, Merewa and Jiren site) should be used for appropriate installation type in order to save the installed pipe.
- Since Seka site fulfills the criteria for installation Type II, III and Type IV it is better to use for backfill materials over the top of reinforced concrete pipe.

- Further detailed investigation on settlement analysis of foundation soil under the pipe due to traffic and earth load should be made.

- The fill height over the top of reinforced concrete pipe used on the design was according to the standard specification and it was common for all sites but from analysis for each site there was specific backfill height. Therefore, it is better if specific backfill height for each site has to be used.

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APPENDIX

Appendix A: Figures of standard installation and tables of specifications

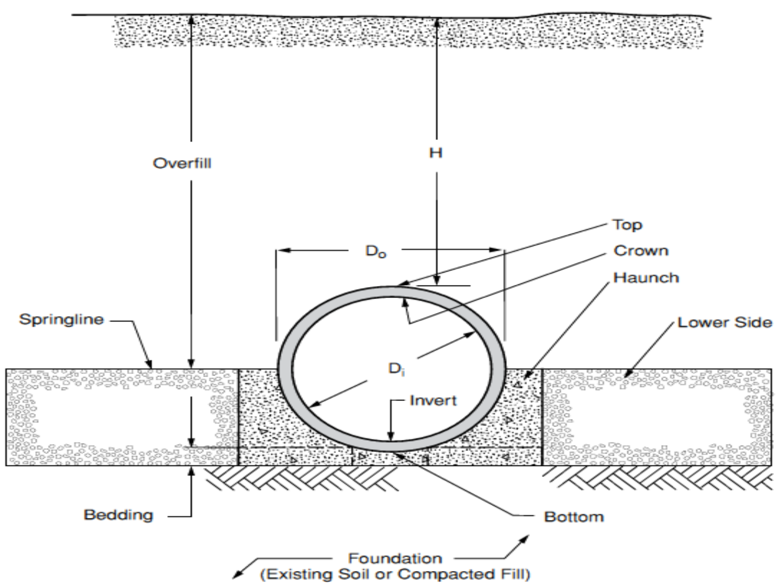


Figure A.1Pipe /Installation Terminology

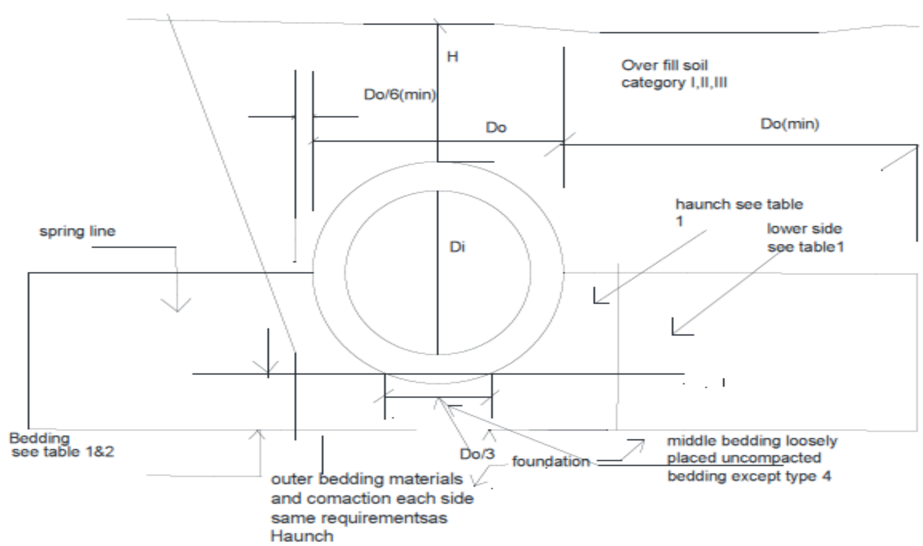


Figure A.2Standard Trench/Embankment Installation

Table A.1.Bedding and Compaction Requirements for Reinforced Concrete Pipe (per ASTM C 1479)1

Installation type	Bedding thickness	Compaction requirements (Minimum Standard Proctor %)1					
		Haunch and outer bedding			Lower side bedding or Undisturbed Earth equivalent		
		Gravelly Sand2	Sandy Silt3	Silty Clay4	Gravelly Sand2	Sandy Silt3	Silty Clay4
1	Do/24 in. minimum; not less than 3 in. If rock foundation, use Do/12 minimum; not less than 6 in.	95	N/a	n/a	90	95	100
2	Do/24 in. minimum; not less than 3 in. If rock foundation, use Do/12 minimum; not less than 6 in.	90	95	n/a	85	90	95
3	Do/24 in. minimum; not less than 3 in. If rock foundation, use Do/12 in. minimum; not less than 6 in.	85	90	95	85	90	95
4	No bedding required, except if rock foundation, use Do/12 in. minimum; not less than 6 in.	No	None	90	None	None	85

Note1 the backfill requirements recommended in Section 2.2.4 to meet the service life requirements are similar to installation Types I and II with an increased standard of compaction to 95% Standard Proctor density.

2 SW or GW material (ASTM D 2487) or A-1 or A-3 (AASHTO M 145). Uniformly graded coarse-grained soils (GP, SP or A-3) shall only be used if provisions are made to evaluate and control possible migration of fines into open voids. Pea gravel shall not be used.

3 ML, SM or GM material (ASTM D 2487) or A-4, A-2-4 or A-2-5 (AASHTO M 145)

4 CL, GC or SC (ASTM D 2487) or A-2-6 or A-2-7, A-5, A-6 (AASHTO M 145)

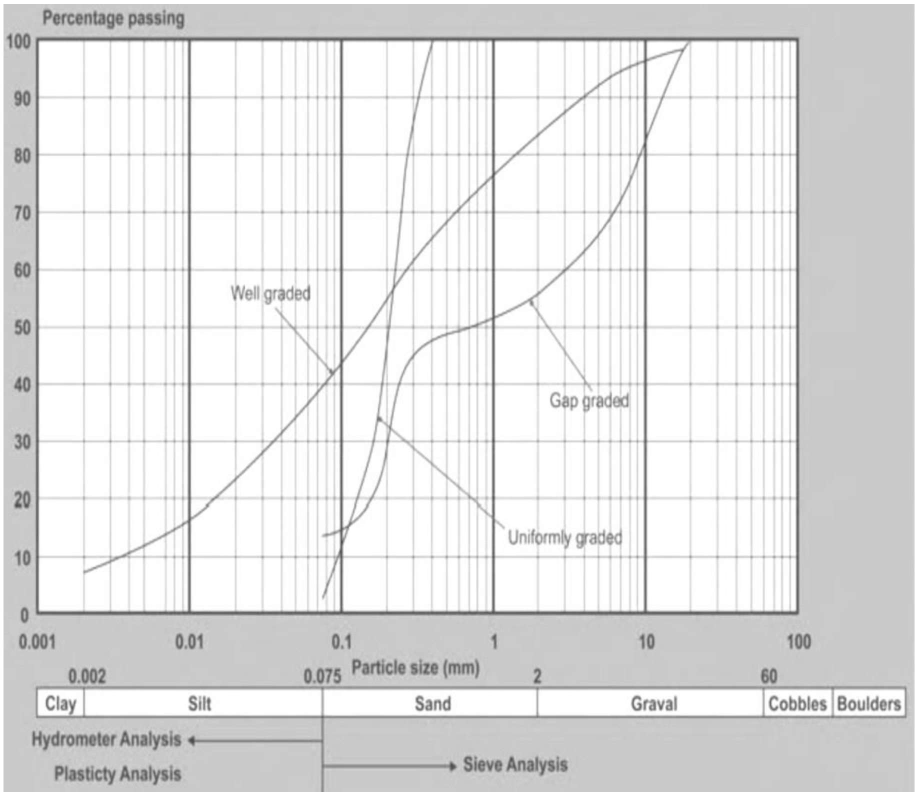
Table A.2`Unified soil classification system

Coarse Grained soils have less than 50% passing the # 200 sieve:				
Symbol	passing #200	Cu = D60/D10	Cc = (D30)2/D10xD60	soil Description
GW	<5%	4 or higher	1 to 3	well graded
GP	<5%	Less than 4	1 to 3	poorly graded
GW - GM	5 to 12%	4 or higher	1 to 3 but with <15% sand	well-graded with silt
GW - GM	5 to 12%	4 or higher	1 to 3 but with =>15% sand	well graded gravel with silt
GW-GC	5 to 12%	4 or higher	1 to 3 but with<15% sand	well graded gravel with clay or silty clay
GW-GC	5 to 12%	4 or higher	1 to 3 but with15% sand	well graded gravel with clay and sand
GC	>12%	N/A	N/A ,<15% sand	clayey Gravel
GC	>12%	N/A	N/A ,>15% sand	clayey Gravel with sand
GM-GC	>12%	N/A	N/A ,<15% sand	clay silt with gravel
GM-GC	12%	N/A	N/A,>15% sand	clay silt with sand
SW	<5%	6 or higher	1 to 3	well graded sand
SP	<5%	Less than 6	1 to 3	poorly graded
SM	>12%	N/A	N/A	silty sand or sandy silt
SC	>12%	N/A	N/A	Claey sand or sandy clay
SC-SM	>12%	N/A	N/A	silty clay with sand

Where: D10, D30, and D60 are the grain size diameter corresponding to 10%, 30% and 60% passing screen and N/A (not accepted).

Cu = Uniformity Coefficient; gives the range of grain sizes in a given sample. Higher Cu means well graded.

Cc= Coefficient of Curvature is a measure of the smoothness of the gradation curve.



FigureA.3grading curve by Unified soil classification [6]

Table A.3.Bedding Factors, BfLL, for HS20 Live Loadings [ ]

Fill height in Ft	pipe diameter inches										
	12	24	36	48	60	72	84	96	108	120	144
0.5	2.2	1.7	1.4	1.3	1.3	1.1	1.1	1.1	1.1	1.1	1.1
1	2.2	2.2	1.7	1.5	1.4	1.3	1.3	1.3	1.1	1.1	1.1
1.5	2.2	2.2	2.2	1.8	1.5	1.4	1.4	1.3	1.3	1.3	1.1
2	2.2	2.2	2.2	2	1.8	1.5	1.5	1.4	1.4	1.3	1.3
2.5	2.2	2.2	2.2	2.2	2	1.8	1.7	1.5	1.4	1.4	1.3
3	2.2	2.2	2.2	2.2	2.2	2.2	1.8	1.7	1.5	1.5	1.4
3.5	2.2	2.2	2.2	2.2	2.2	2.2	1.9	1.8	1.7	1.5	1.4
4	2.2	2.2	2.2	2.2	2.2	2.2	2.1	1.9	1.8	1.7	1.5
4.5	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2	1.9	1.8	1.7
5	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2	1.9	1.8

Table A.4.Concrete Pipe for Shallow Cover Installations of minimum

pipe Diameter	Minimum cover in meters				
	Plain AASHTO 86M	class II AASHTO 170M	class III AASHTO 170M	class IV AASHTO 170M	class V AASHTO 170M
300	50	0.45	0.45	0.3	0.15
450	63	0.45	0.45	0.3	0.15
600	75	0.45	0.45	0.3	0.15
750	88	0.45	0.45	0.3	0.15
900	100	0.45	0.45	0.3	0.15
1200	125		0.45	0.3	0.15
1500	150		0.45	0.3	0.15
1800	175		0.45	0.3	0.15
2100	200		0.45	0.3	0.15
			0.45	0.3	0.15

Table A.5.Concrete Pipe for Shallow Cover Installations maximum

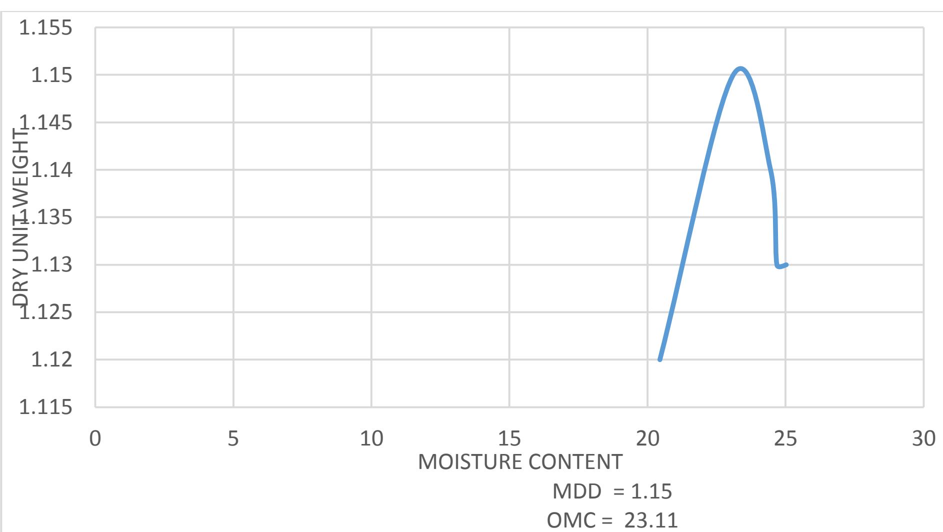
pipe Diameter	Maximum cover in meters				
	Plain AASHTO 86M	class II AASHTO 170M	class III AASHTO 170M	class IV AASHTO 170M	class V AASHTO 170M
300	5.5	3	4.3	6.5	7.9
450	5.5	3.4	4.3	6.5	9
600	5	3.4	4.6	6.5	9
750		3.4	4.6	7	9
900		3.4	4.6	7	9
1200		3.7	4.9	7	9
1500		3.7	4.9	7.5	9
1800		3.7	4.9	7.5	9
2100		3.7	4.9	7.5	9

Appendix B: Compaction test Analysis data

Location: Jiren quarry site at position 1
Sample type: Disturbed
Test type: standard proctor

Trial number	1	2	3	4	5
Weight of soil +Mold	4280	4340	4345	4327	4335
Weight of Mold	3005	3005	3005	3005	3005
Weight of soil	1275	1335	1340	1322	1330
Volume of Mold, cc	944	944	944	944	944
Wet density of soil	1.35	1.41	1.42	1.4	1.41
Container number	p44	E	D	D2	G2
Wet soil + container, g	96.1	122.4	113	102.24	102.57
Dry soil +container, g	84.56	104.63	94.43	85.92	85.93
Weight of water, g	11.63	17.77	18.75	16.32	16.64
Weight of container, g	27.7	27.74	17.76	17.49	17.55
Weight of dry soil, g	56.86	76.89	76.67	68.43	68.38
Moisture content %	20.45	23.11	24.46	23.85	25.03
Dry density of soil, g/cc	1.12	1.15	1.14	1.13	1.13

Jiren -compaction test result at position 1



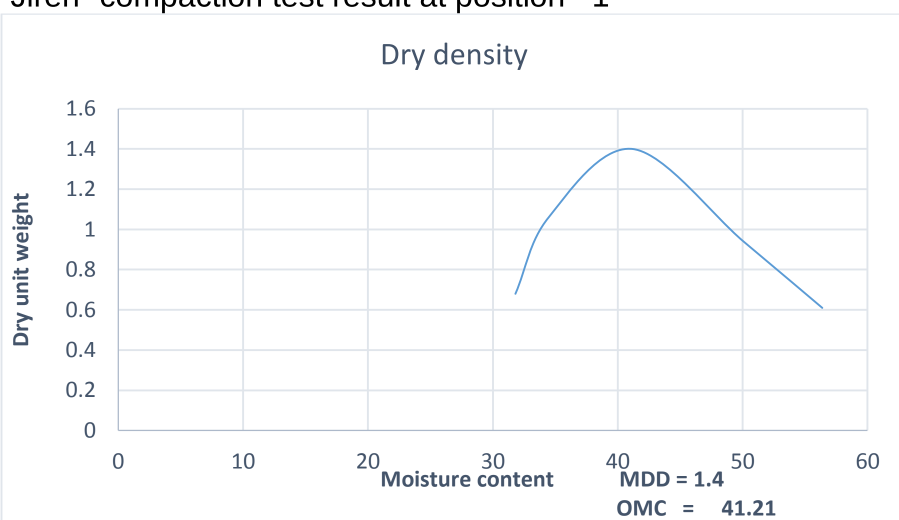
Location: Jiren quarry site at position 2

Sample type: Disturbed

Test type: standard proctor

Trial number	1	2	3	4	5
Weight of soil +Mold	3850	4540	4864	4338	3990
Weight of Mold	3005	3005	3005	3005	3005
Weight of soil	845	1337	1859	1333	1360
Volume of Mold, cc	944	944	944	944	944
Wet density of soil	0.9	1.42	1.97	1.41	1.04
Container number	p44	E	D	G2	D2
Wet soil + container, g	105.08	132.01	127.93	122.66	99.66
Dry soil +container, g	85.43	105.01	95.78	88.42	72.43
Weight of water, g	19.65	26.99	32.15	34.24	27.23
Weight of container, g	23.65	26.7	17.76	20.04	24.12
Weight of dry soil, g	61.78	78.31	78.02	68.38	48.31
Moisture content %	31.81	34.47	41.21	50.07	56.37
Dry density of soil, g/cc	0.68	1.06	1.4	0.94	0.67

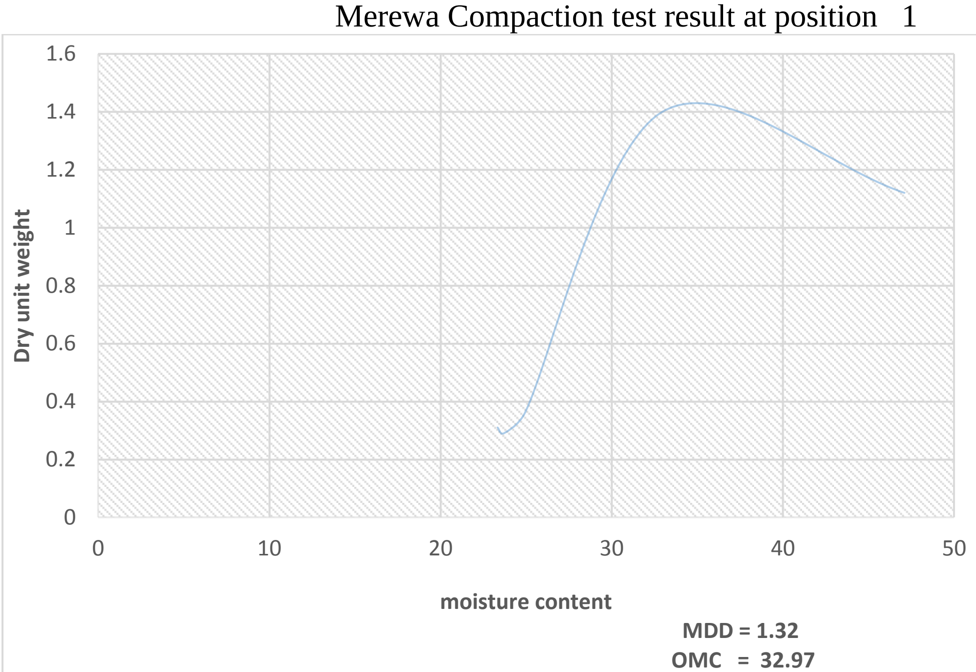
Jiren -compaction test result at position 1



Location: Merewa quarry site at position 1
Sample type: Disturbed
Test type: standard proctor

Trial number	1	2	3	4	5
Weight of soil +Mold, g	3360	3345	3430	4765	4560
Weight of Mold, g	3005	3005	3005	3005	3005
Weight of soil , g	355	340	425	1760	1555
Volume of Mold, cc	944	944	944	944	944
Wet density of soil	0.38	0.36	0.45	1.86	1.65
Container number	1	3	u1	A3	A2
Wet soil + container, g	109.63	106.02	145.95	148.16	132.34
Dry soil +container, g	94.14	89.19	125.8	118.37	95.62
Weight of water, g	15.49	16.83	20.15	29.79	36.72
Weight of container, g	27.78	18.17	44.96	28.01	17.65
Weight of dry soil, g	66.36	71.02	80.84	90.36	77.97
Moisture content %	23.34	23.7	24.93	32.97	47.1
Dry density of soil, g/cc	0.31	0.29	0.36	1.4	1.12

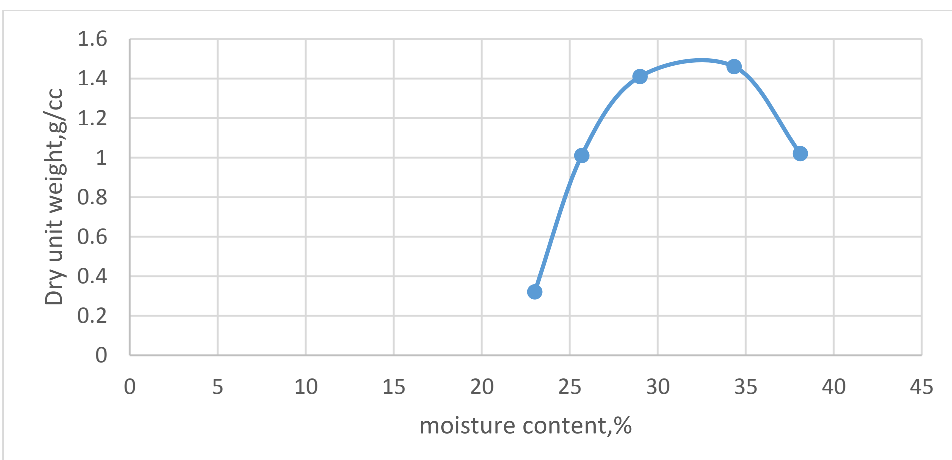
Merewa Compaction test result at position 1



Location: Merewa quarry site at position 2
Sample type: Disturbed
Test type: standard proctor

Trial number	1	2	3	4	5
Weight of soil +Mold, g	3370	3423	4243	4856	4565
Weight of Mold, g	3005	3005	3005	3005	3005
Weight of soil , g	365	418	1238	1.851	1560
Volume of Mold, cc	944	944	944	944	944
Wet density of soil	0.39	0.44	1.31	1.96	1.65
Container number	1	3	u1	A3	A2
Wet soil + container, g	110.12	115.34	147.12	149.45	136.42
Dry soil +container, g	94.45	95.68	130.21	120.49	102.24
Weight of water, g	15.67	19.66	14.34	32.23	34.18
Weight of container, g	26.34	19.12	39.65	29.01	18.56
Weight of dry soil, g	68.1	76.56	41.87	93.86	89.68
Moisture content %	23.01	25.68	29.01	34.34	38.11
Dry density of soil, g/cc	0.32	1.01	1.41	1.46	1.02

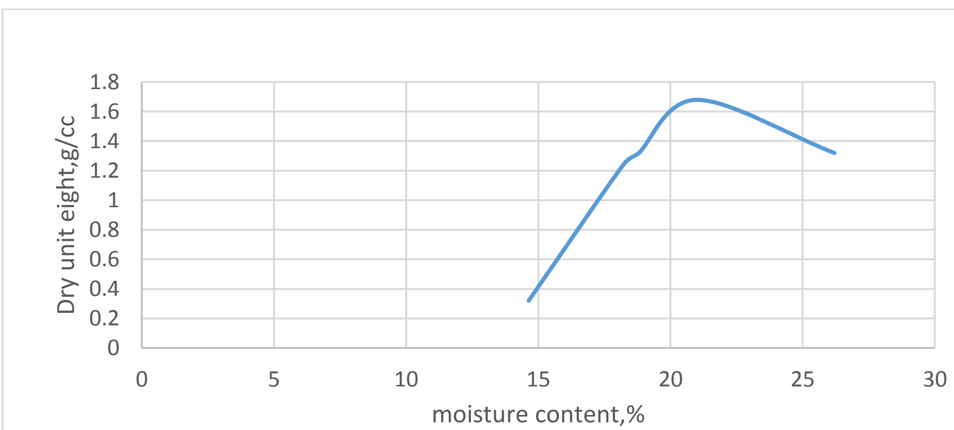
Merewa Compaction test result at position 2



Location: Seka quarry site at position 1
Sample type: Disturbed
Test type: standard proctor

Trial number	1	2	3	4	5
Weight of soil +Mold	3405	3586	4840	4800	4765
Weight of Mold	3005	3005	3005	3005	3005
Weight of soil	400	581	1835	1795	1760
Volume of Mold, cc	944	944	944	944	944
Wet density of soil	0.42	0.62	1.94	1.9	1.86
Container number	1	3	u1	A3	A2
Wet soil + container, g	115.75	155.9	158.43	138.91	156.44
Dry soil +container, g	103.29	137.65	139.04	118.79	133.12
Weight of water, g	12.46	18.25	19.39	20.12	23.32
Weight of container, g	18.14	17.63	14.06	26.94	29.42
Weight of dry soil, g	85.15	120.02	124.98	91.85	103.7
Moisture content %	14.63	15.21	21.23	21.91	22.49
Dry density of soil, g/cc	0.37	0.54	1.68	1.56	1.52

Seka Compaction test result at position 1

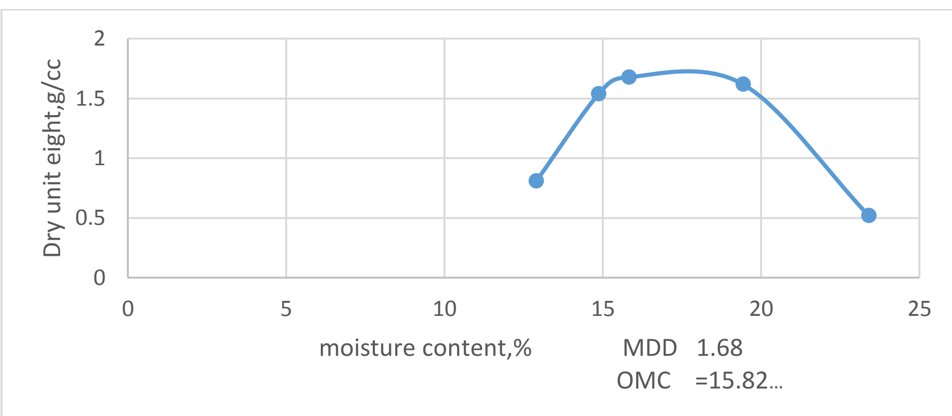


Location: Seka quarry site at position 2

Sample type: Disturbed

Test type: standard proctor

Trial number	1	2	3	4	5
Weight of soil +Mold	3408	3587	4845	4842	3589
Weight of Mold	3005	3005	3005	3005	3005
Weight of soil	403	582	1840	1837	584
Volume of Mold, cc	944	944	944	944	944
Wet density of soil	0.43	0.62	1.95	1.94	0.62
Container number	1	3	u1	A3	A2
Wet soil + container, g	116.75	156.23	157.64	138.9	155.64
Dry soil +container, g	104.3	138.42	138.23	120.51	134.45
Weight of water, g	12.45	17.81	19.41	18.39	21.19
Weight of container, g	17.15	18.64	15.54	25.88	28.42
Weight of dry soil, g	87.15	119.78	122.69	94.63	106.03
Moisture content %	12.9	14.87	15.82	19.43	23.4
Dry density of soil, g/cc	*0.81*	*1.54*	*1.68*	*1.62*	*0.52*



Appendix C: Atterberg limits test results data

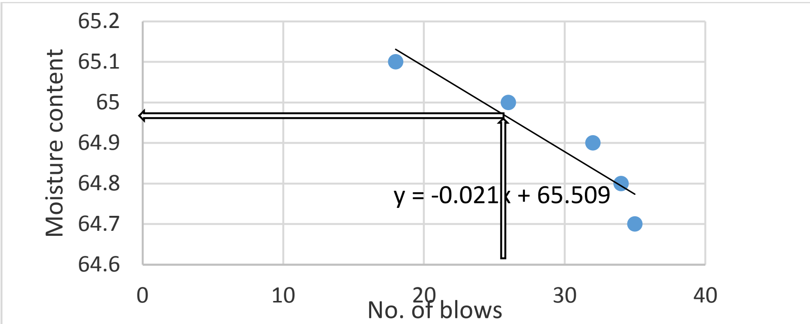
Liquid and plastic Limit Determination — Sample type: Disturbed

Location: Jiren quarry site at position 1 — Test Type: Liquid & Plastic Limit

Determinations, data and computation sheet

Type of test	LIQUID LIMIT				
Test No.	1	2	3	4	5
Number of blows	18	26	32	34	35
Container No.	p41	D4	E	P42	D3
Wet Soil+Cont (g)	52.89	51.1	53.74	48.91	62.68
Dry Soil+Cont (g)	43.03	38.03	39.35	36.84	49.08
Mass Container (g)	27.88	17.69	17.13	18.1	27.86
Mass Moisture (g)	9.9	13.1	14.4	12.1	13.6
Mass Dry Soil (g)	15.2	20.3	22.2	18.7	21.2
Moist Content (%)	65.1	65	64.9	64.7	64.2

Type of test	PLASTIC LIMIT	
Test No.	1	2
Number of blows		
Container No.	U2	A5R
Wet Soil+Cont (g)	37.91	40.69
Dry Soil+Cont (g)	34.77	36.4
Mass Container (g)	28.1	27.7
Mass Moisture (g)	3.1	4.3
Mass Dry Soil (g)	6.7	8.7
Moist Content (%)	46	49



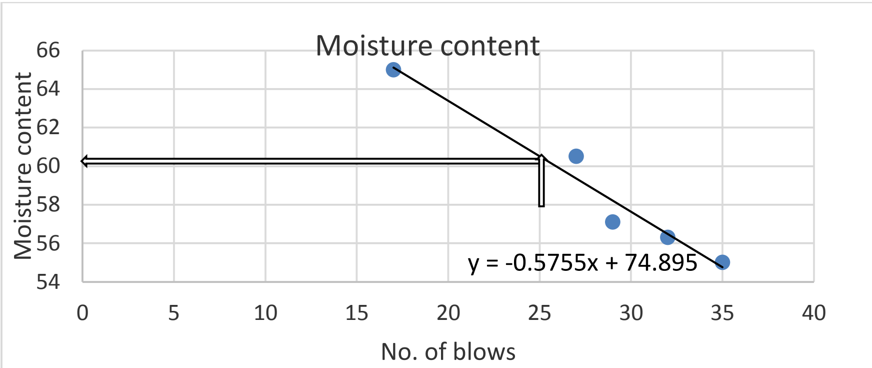
Liquid and plastic Limit Determination Sample type: Disturbed

Location: Jiren quarry site at position 2 Test Type: Liquid & Plastic Limit

Determinations, data and computation sheet

Type of test	LIQUID LIMIT				
Test No.	1	2	3	4	5
Number of blows	17	27	29	32	35
Container No.	p41	D4	E	P42	D3
Wet Soil+Cont (g)	53.72	62.58	58.45	51.7	49.2
Dry Soil+Cont (g)	39.3	49.24	44.62	39.6	38.03
Mass Container (g)	17.1	25.04	20.5	18.1	17.69
Mass Moisture (g)	14.42	13.3	13.8	12.1	11.17
Mass Dry Soil (g)	22.2	22	24.1	21.5	20.3
Moist Content (%)	65.0	60.5	57.1	56.3	55

type of test	PLASTIC LIMIT	
Test No.	1	2
Number of blows		
Container No.	U2	A5R
Wet Soil+Cont (g)	39.52	40.3
Dry Soil+Cont (g)	35.42	35.6
Mass Container (g)	26.2	24.6
Mass Moisture (g)	4.1	4.7
Mass Dry Soil (g)	9.2	11
Moist Content (%)	45	43



Liquid and plastic Limit Determination

Sample type: Disturbed

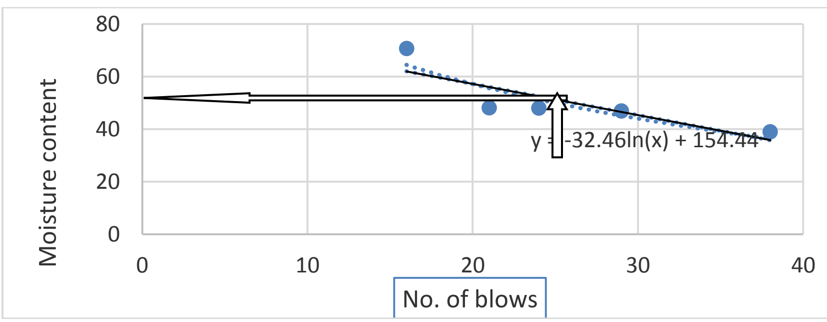
Location: Merewa quarry site at position 1

Test Type: Liquid & Plastic Limit

Determinations, data and computation sheet

Type of test	LIQUID LIMIT				
Test No.	1	2	3	4	5
Number of blows	16	21	24	29	38
Container No.	A1	A2	A3	G1	B2
Wet Soil+Cont (g)	44.06	41.01	39.41	37.8	37.2
Dry Soil+Cont (g)	35.02	34.1	32.4	30.99	31.5
Mass Container (g)	22.24	19.76	17.79	16.49	17.01
Mass Moisture (g)	9.04	6.91	7.01	6.8	5.7
Mass Dry Soil (g)	12.78	14.34	14.61	14.5	14.49
Moist Content (%)	70.7	48.2	48.0	46.9	39

Type of test	PLASTIC LIMIT	
Test No.	1	2
Container No.	B3	B1
Wet Soil+Cont (g)	33.55	45.15
Dry Soil+Cont (g)	29.6	40.99
Mass Container (g)	17.76	27.78
Mass Moisture (g)	4	4.2
Mass Dry Soil (g)	11.8	13.2
Moist Content (%)	34	32



Liquid and plastic Limit Determination

Sample type: Disturbed

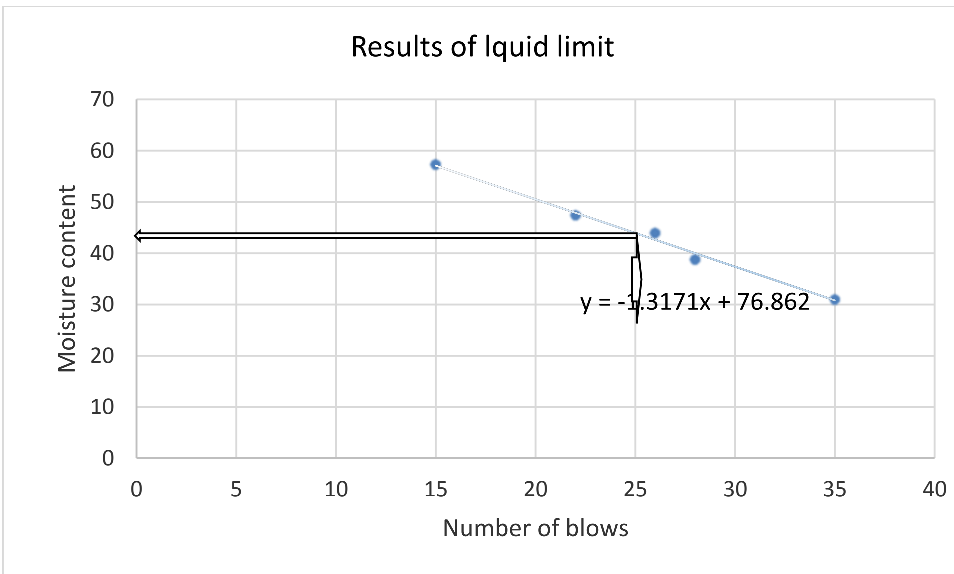
Location: Merewa quarry site at position 2

Test Type: Liquid & Plastic Limit

Determinations, data and computation sheet

Type of test	LIQUID LIMIT				
Test No.	1	2	3	4	5
Number of blows	15	22	26	28	35
Container No.	B1	B2	B3	B4	B5
Wet Soil+Cont (g)	45.01	42.2	40.1	38.4	37.5
Dry Soil+Cont (g)	37.2	35	32.8	32.51	33.12
Mass Container (g)	23.56	19.76	18.01	17.34	17.01
Mass Moisture (g)	7.81	7.2	7.3	5.9	4.4
Mass Dry Soil (g)	13.64	15.2	14.8	15.2	14.49
Moist Content (%)	57.3	47.4	49.4	38.8	31

Type of test	PLASTIC LIMIT	
Test No.	1	2
Container No.	B3	B1
Wet Soil+Cont (g)	33.55	45.21
Dry Soil+Cont (g)	29.8	39.45
Mass Container (g)	17.76	26.56
Mass Moisture (g)	3.8	5.8
Mass Dry Soil (g)	12.04	12.9
Moist Content (%)	32	45



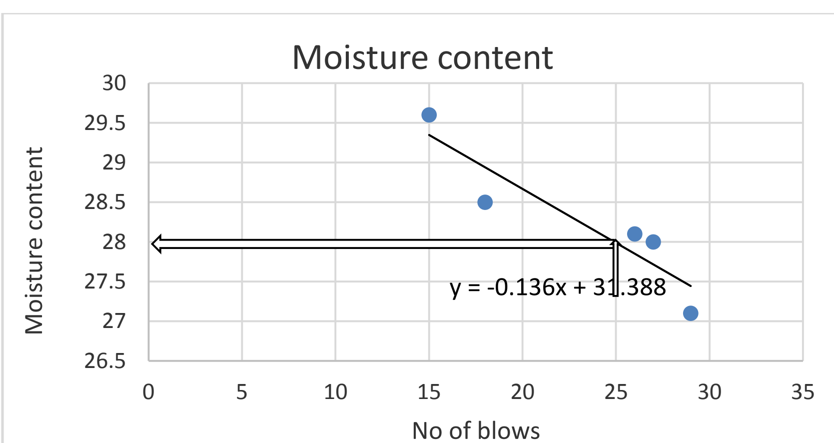
Liquid and plastic Limit Determination Sample type: Disturbed

Location: Seka quarry site at position 1 Test Type: Liquid & Plastic Limit

Determinations, data and computation sheet

Type of test	LIQUID LIMIT				
Test No.	1	2	3	4	5
Number of blows	15	18	26	27	29
Container No.	A3	515	P43	M3	B2
Wet Soil+Cont (g)	74	71.78	69.38	70.19	82.08
Dry Soil+Cont (g)	63.48	62	60.05	60.95	70.54
Mass Container (g)	28	27.6	26.95	27.88	28.11
Mass Moisture (g)	10.5	9.8	9.3	9.2	11.5
Mass Dry Soil (g)	35.5	34.4	33.1	33.1	42.4
Moist Content (%)	29.6	28.5	28.1	28	27.1

Type of test	PLASTIC LIMIT	
Test No.	1	2
Container No.	G1	V1
Wet Soil+Cont (g)	38.54	39.38
Dry Soil+Cont (g)	34.46	35.11
Mass Container (g)	18.02	18
Mass Moisture (g)	4.1	4.3
Mass Dry Soil (g)	16.4	17.1
Moist Content (%)	25	25



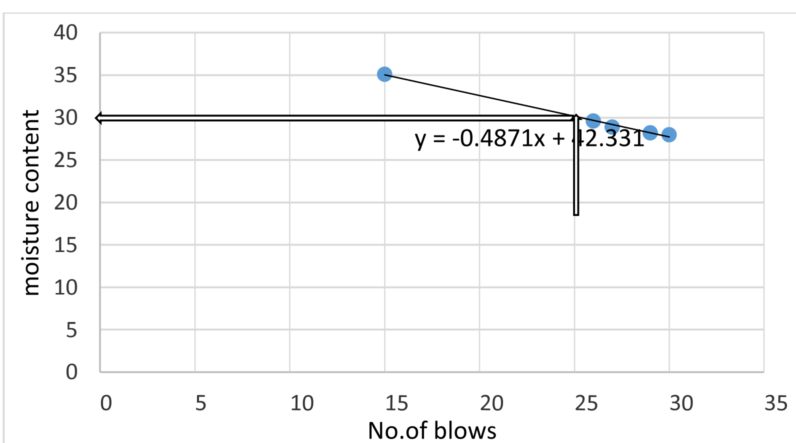
Liquid and plastic Limit Determination Sample type: Disturbed

Location: Seka quarry site at position 2 Test Type: Liquid & Plastic Limit

Determinations, data and computation sheet

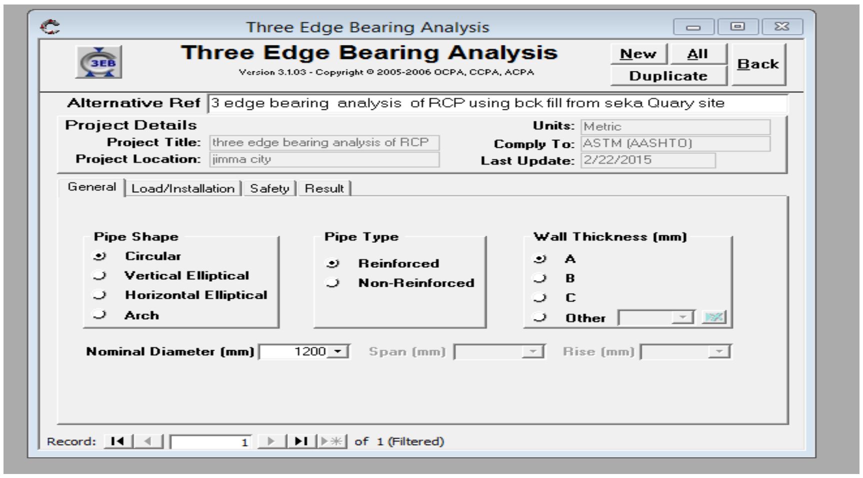
Type of test		LIQUID LIMIT				
Test No.		1	2	3	4	5
Number of blows		15	26	27	29	30
Container No.		A1	A2	A3	A4	A5
Wet Soil+Cont (g)		76.52	71.9	82.09	69.5	70.17
Dry Soil+Cont (g)		63.48	61.9	69.98	60.15	60.92
Mass Container (g)		26.5	27.6	28.1	26.93	27.77
Mass Moisture (g)		13	10	12.1	9.4	9.3
Mass Dry Soil (g)		37	33.8	41.9	33.2	33.2
Moist Content (%)		35.1	29.6	28.9	28.2	28

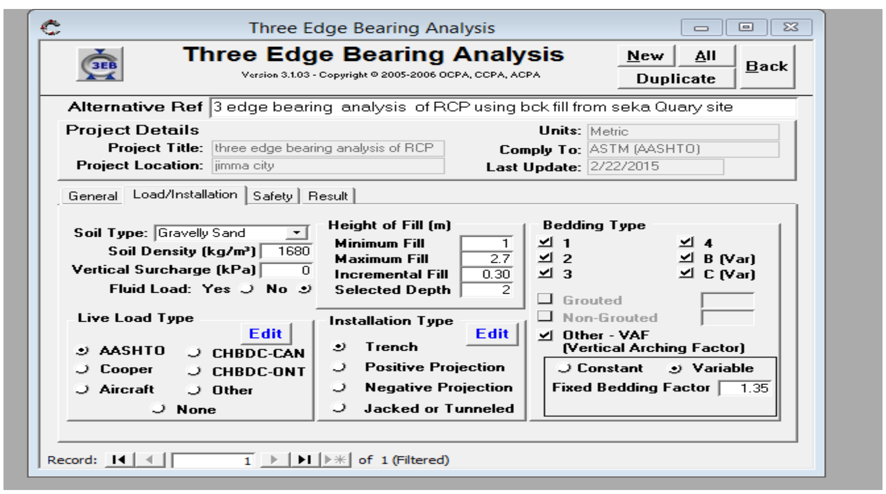
Type of test	PLASTIC LIMIT	
Test No.	1	2
Container No.	G1	V1
Wet Soil+Cont (g)	38.55	39.36
Dry Soil+Cont (g)	34.46	35.1
Mass Container (g)	18.02	18.03
Mass Moisture (g)	4.09	4.26
Mass Dry Soil (g)	16.4	17.07
Moist Content (%)	25	25

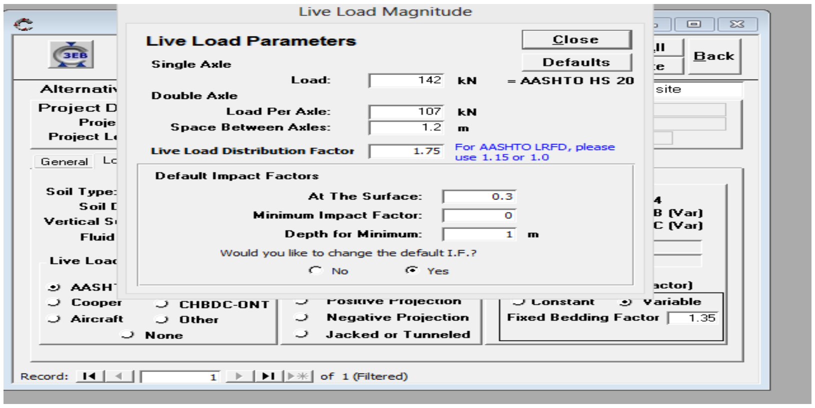


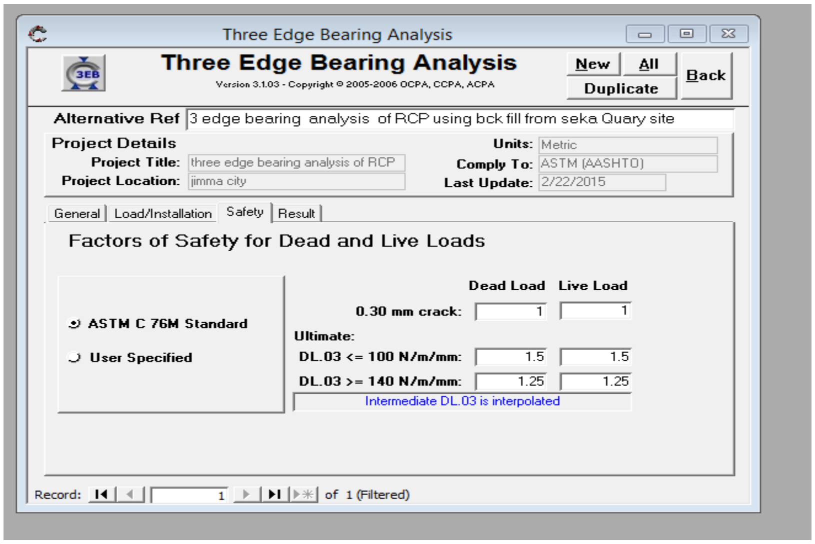
Appendix D:Three edge bearing analysis dialogue boxes

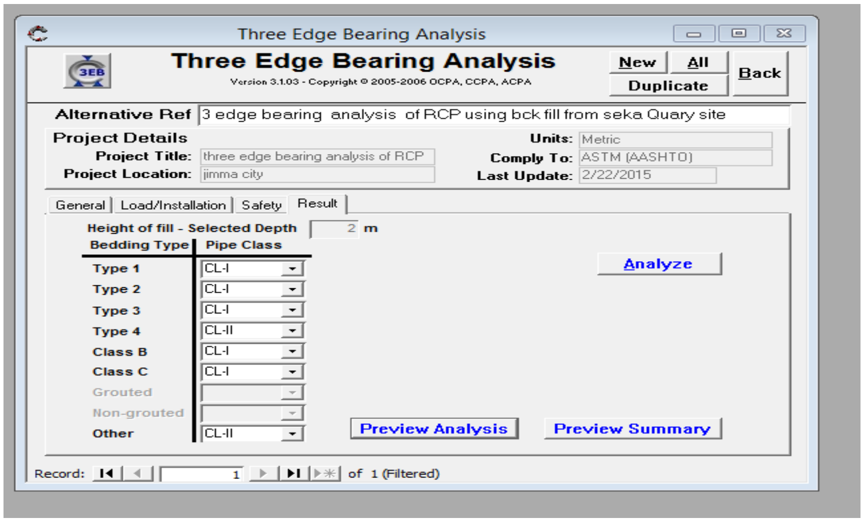
The following dialogue box shows procedural three edge bearing analysis of concrete pipe with pipe Pac software



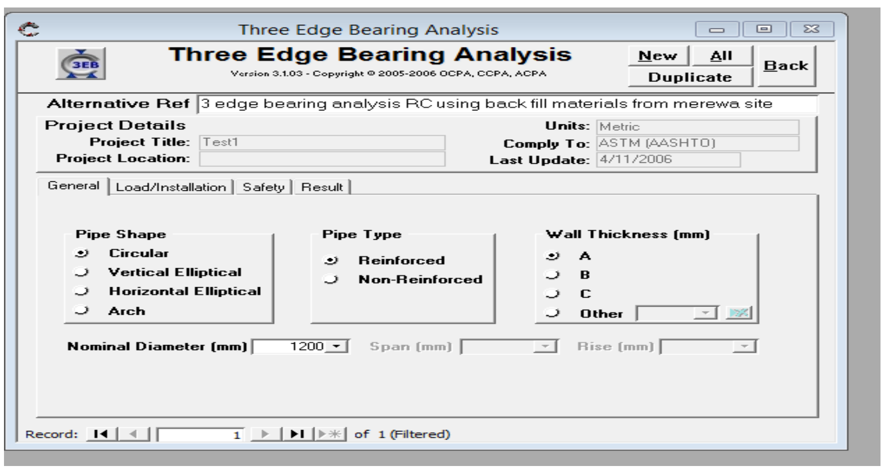


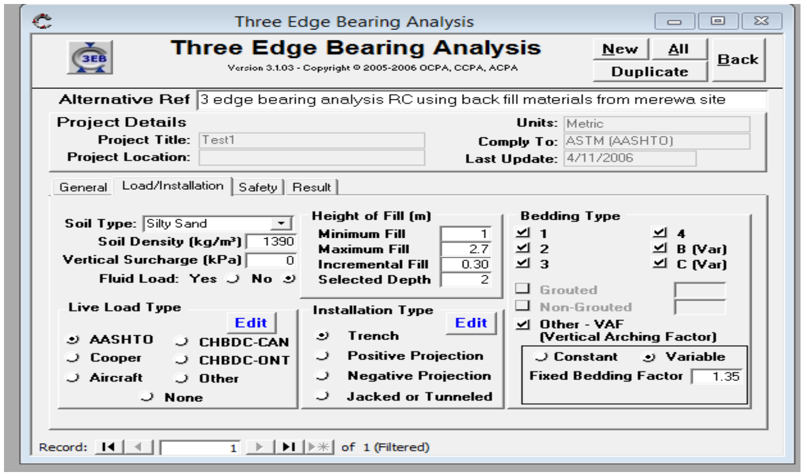

Live Load Magnitude
Live Load Parameters
Close
Defaults
Single Axle
Load: 142 kN = AASHTO HS 20
Double Axle
Load Per Axle: 107 kN
Space Between Axles: 1.2 m
Live Load Distribution Factor 1.75
For AASHTO LRFD, please use 1.15 or 1.0
Default Impact Factors
At The Surface: 0.3
Minimum Impact Factor: 0
Depth for Minimum: 1 m
Would you like to change the default I.F.?
No
Yes
Back
Cooper
CHBDC-ONT
Aircraft
Other
None
Negative Projection
Jacked or Tunneled
Constant
Variable
Fixed Bedding Factor 1.35
Record: 1 of 1 (Filtered)

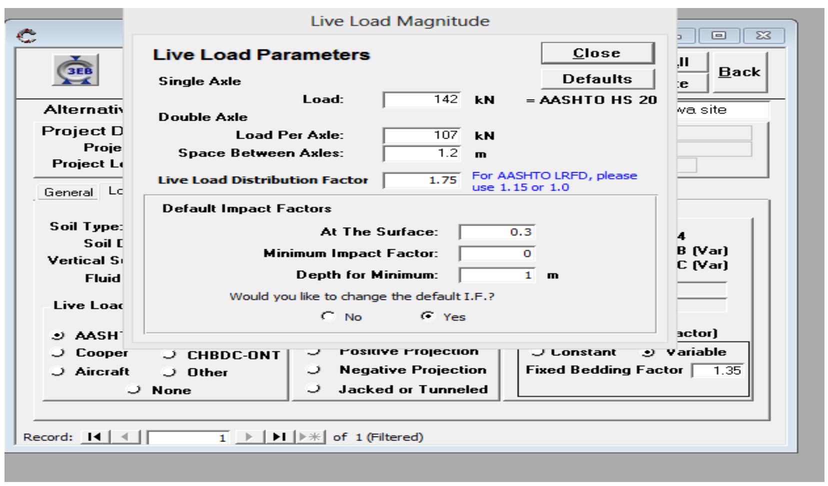

Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref 3 edge bearing analysis of RCP using bck fill from seka Quary site
Project Details
Project Title: three edge bearing analysis of RCP
Project Location: jimma city
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 2/22/2015
General
Load/Installation
Safety
Result
Factors of Safety for Dead and Live Loads
ASTM C 76M Standard
User Specified
Dead Load
Live Load
0.30 mm crack: 1 1
Ultimate:
DL.03 <= 100 N/m/mm: 1.5 1.5
DL.03 >= 140 N/m/mm: 1.25 1.25
Intermediate DL.03 is interpolated
Record: 1 of 1 (Filtered)

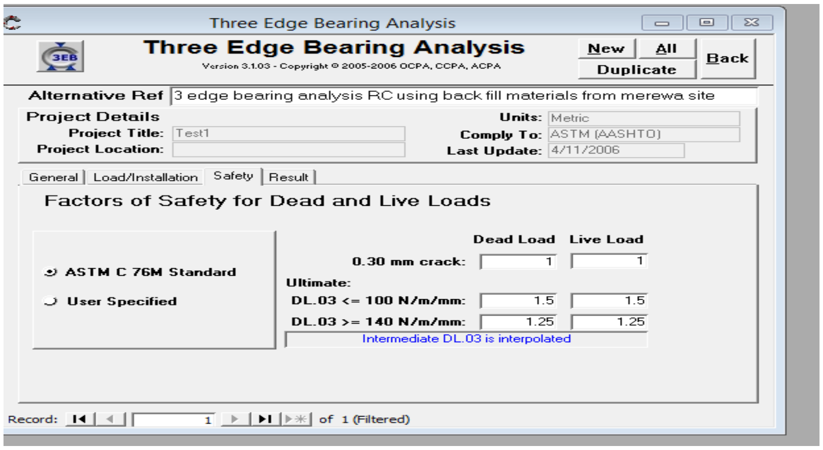


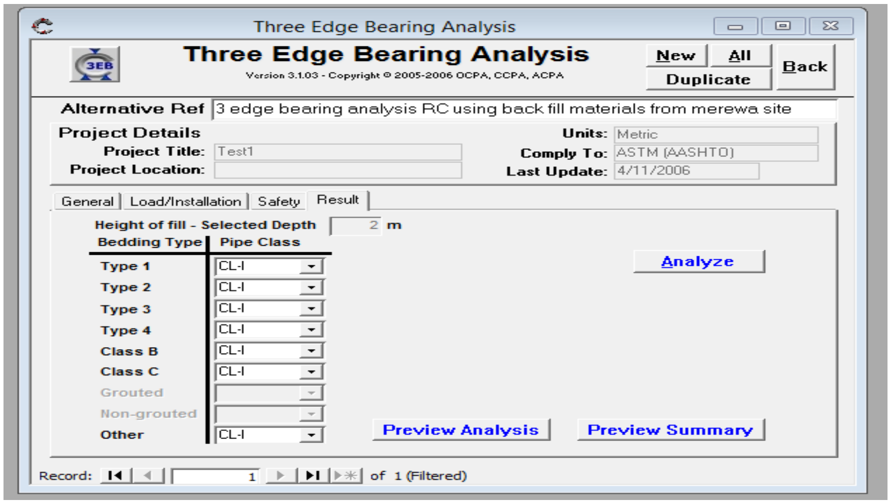
The following dialogue box shows the procedural three edge analysis of Merewa site

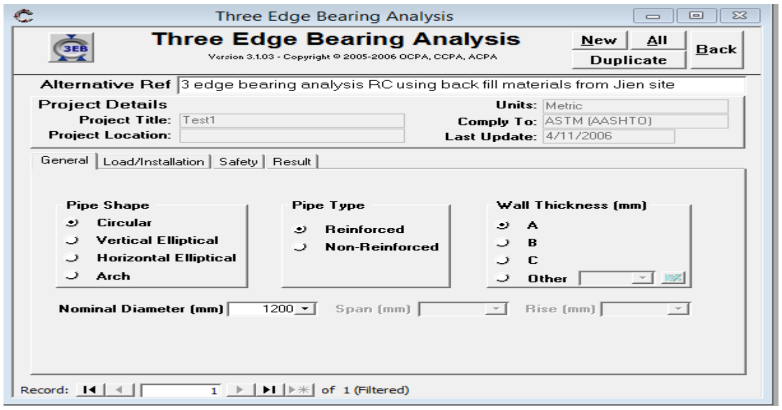


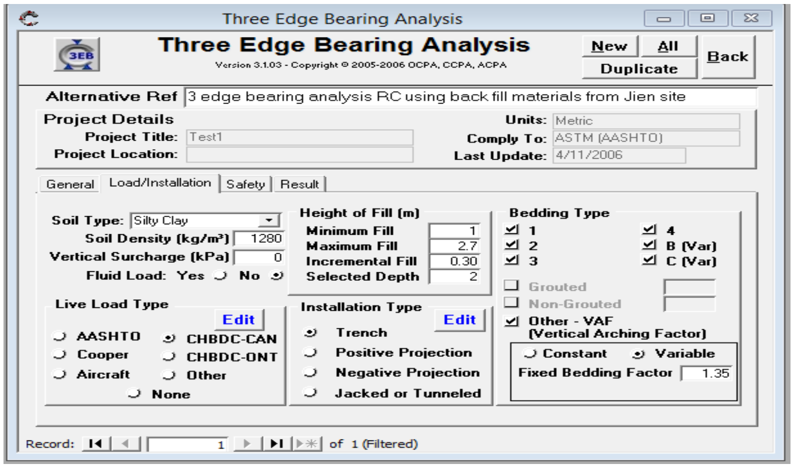

Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref
3 edge bearing analysis RC using back fill materials from merewa site
Project Details
Project Title: Test1
Project Location:
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 4/11/2006
General
Load/Installation
Safety
Result
Soil Type: Silty Sand
Soil Density (kg/m³) 1390
Vertical Surcharge (kPa) 0
Fluid Load: Yes No
Height of Fill (m)
Minimum Fill 1
Maximum Fill 2.7
Incremental Fill 0.30
Selected Depth 2
Bedding Type
1
2
3
4
B (Var)
C (Var)
Grouted
Non-Grouted
Other - VAF
(Vertical Arching Factor)
Constant
Variable
Fixed Bedding Factor 1.35
Live Load Type
Edit
AASHTO
CHBDC-CAN
Cooper
CHBDC-ONT
Aircraft
Other
None
Installation Type
Edit
Trench
Positive Projection
Negative Projection
Jacked or Tunneled
Record: 1 of 1 (Filtered)

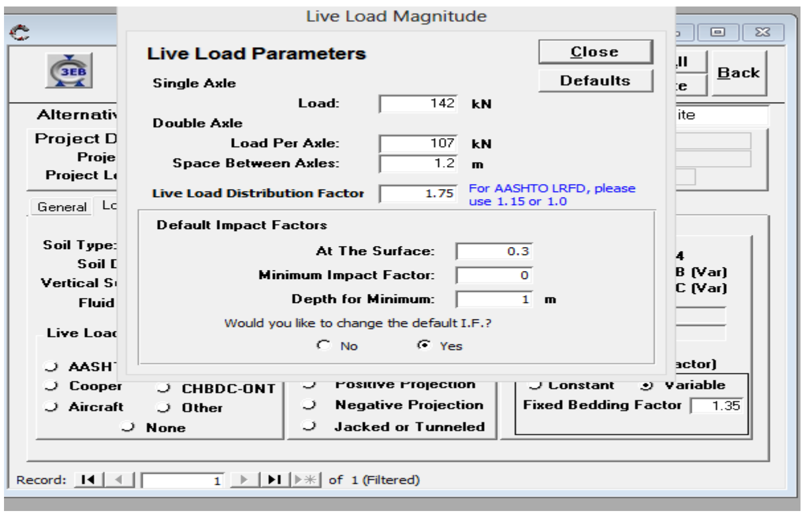

Live Load Magnitude
Live Load Parameters
Close
Defaults
Single Axle
Load: 142 kN = AASHTO HS 20
Double Axle
Load Per Axle: 107 kN
Space Between Axles: 1.2 m
Live Load Distribution Factor 1.75
For AASHTO LRFD, please use 1.15 or 1.0
Default Impact Factors
At The Surface: 0.3
Minimum Impact Factor: 0
Depth for Minimum: 1 m
Would you like to change the default I.F.?
No
Yes
Back
Fixed Bedding Factor 1.35
Record: 1 of 1 (Filtered)

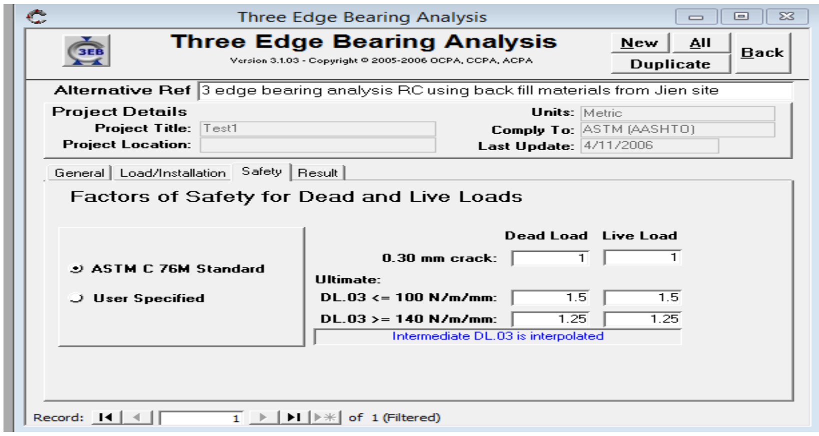

Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref 3 edge bearing analysis RC using back fill materials from merewa site
Project Details
Project Title: Test1
Project Location:
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 4/11/2006
General
Load/Installation
Safety
Result
Factors of Safety for Dead and Live Loads
ASTM C 76M Standard
User Specified
Dead Load
Live Load
0.30 mm crack: 1 1
Ultimate:
DL.03 <= 100 N/m/mm: 1.5 1.5
DL.03 >= 140 N/m/mm: 1.25 1.25
Intermediate DL.03 is interpolated
Record: 1 of 1 (Filtered)

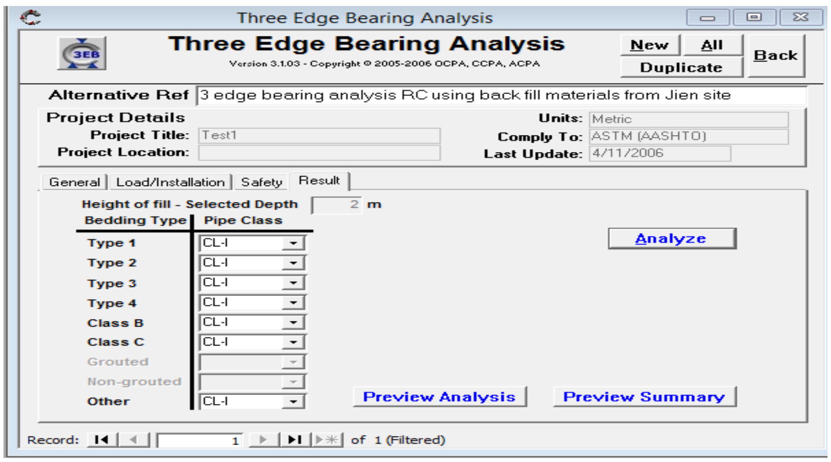

Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref 3 edge bearing analysis RC using back fill materials from merewa site
Project Details
Project Title: Test1
Project Location:
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 4/11/2006
General
Load/Installation
Safety
Result
Height of fill - Selected Depth 2 m
Bedding Type
Pipe Class
Type 1 CL-I
Type 2 CL-I
Type 3 CL-I
Type 4 CL-I
Class B CL-I
Class C CL-I
Grouted
Non-grouted
Other CL-I
Analyze
Preview Analysis
Preview Summary
Record: 1 of 1 (Filtered)


Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref 3 edge bearing analysis RC using back fill materials from Jien site
Project Details
Project Title: Test1
Project Location:
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 4/11/2006
General
Load/Installation
Safety
Result
Pipe Shape
Circular
Vertical Elliptical
Horizontal Elliptical
Arch
Pipe Type
Reinforced
Non-Reinforced
Wall Thickness (mm)
A
B
C
Other
Nominal Diameter (mm) 1200
Span (mm)
Rise (mm)
Record: 1 of 1 (Filtered)


Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref
3 edge bearing analysis RC using back fill materials from Jien site
Project Details
Project Title: Test1
Project Location:
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 4/11/2006
General
Load/Installation
Safety
Result
Soil Type: Silty Clay
Soil Density (kg/m³) 1280
Vertical Surcharge (kPa) 0
Fluid Load: Yes No
Height of Fill (m)
Minimum Fill 1
Maximum Fill 2.7
Incremental Fill 0.30
Selected Depth 2
Bedding Type
1
2
3
4
B (Var)
C (Var)
Grouted
Non-Grouted
Other - VAF
(Vertical Arching Factor)
Constant
Variable
Fixed Bedding Factor 1.35
Live Load Type
Edit
AASHTO
CHBDC-CAN
Cooper
CHBDC-ONT
Aircraft
Other
None
Installation Type
Edit
Trench
Positive Projection
Negative Projection
Jacked or Tunneled
Record: 1 of 1 (Filtered)


Live Load Magnitude
Live Load Parameters
Close
Defaults
Single Axle
Load: 142 kN
Double Axle
Load Per Axle: 107 kN
Space Between Axles: 1.2 m
Live Load Distribution Factor 1.75
For AASHTO LRFD, please use 1.15 or 1.0
Default Impact Factors
At The Surface: 0.3
Minimum Impact Factor: 0
Depth for Minimum: 1 m
Would you like to change the default I.F.?
No
Yes
Back
Record: 1 of 1 (Filtered)


Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref
3 edge bearing analysis RC using back fill materials from Jien site
Project Details
Project Title: Test1
Project Location:
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 4/11/2006
General
Load/Installation
Safety
Result
Factors of Safety for Dead and Live Loads
ASTM C 76M Standard
User Specified
Dead Load
Live Load
0.30 mm crack: 1 1
Ultimate:
DL.03 <= 100 N/m/mm: 1.5 1.5
DL.03 >= 140 N/m/mm: 1.25 1.25
Intermediate DL.03 is interpolated
Record: 1 of 1 (Filtered)


Three Edge Bearing Analysis
Three Edge Bearing Analysis
Version 3.1.03 - Copyright © 2005-2006 OCPA, CCPA, ACPA
New
All
Duplicate
Back
Alternative Ref
3 edge bearing analysis RC using back fill materials from Jien site
Project Details
Project Title: Test1
Project Location:
Units: Metric
Comply To: ASTM (AASHTO)
Last Update: 4/11/2006
General
Load/Installation
Safety
Result
Height of fill - Selected Depth 2 m
Bedding Type
Pipe Class
Type 1 CL-I
Type 2 CL-I
Type 3 CL-I
Type 4 CL-I
Class B CL-I
Class C CL-I
Grouted
Non-grouted
Other CL-I
Analyze
Preview Analysis
Preview Summary
Record: 1 of 1 (Filtered)

Appendix E: stress analysis using Bousssinesq theory

The following figures shows the stress analysis using Bousssinesq theory for Seka site

x	y	r	z	r/z	(r/z)2	I δ	Q(kN)	$\delta z_{(kN/}m^2$
0	3.5	3.5	0.0001	35000	1.225E+09	9.00E-24	98	8.82E-14
0.5	3.5	3.535	0.1001	35.31	1247.127	9.00E-09	98	8.80E-05
1	3.5	3.64	0.2001	18.19	330.90901	2.00E-07	98	4.90E-04
1.5	3.5	3.808	0.3001	12.69	161.01335	1.00E-06	98	1.09E-03
2	3.5	4.031	0.4001	10.07	101.50525	5.00E-06	98	3.06E-03
2.5	3.5	4.301	0.5001	8.6	73.964815	1.00E-05	98	3.92E-03
3	3.5	4.61	0.6001	7.682	59.013938	2.00E-02	98	5.44E+00
3.5	3.5	4.95	0.7001	7.07	49.990818	3.00E-05	98	6.00E-03
4	3.5	5.315	0.8001	6.643	44.128381	3.00E-05	98	4.59E-03
4.5	3.5	5.701	0.9001	6.334	40.116271	4.00E-05	98	4.84E-03
5	3.5	6.103	1.0001	6.102	37.239161	5.00E-05	98	4.90E-03
5.5	3.5	6.519	1.1001	5.926	35.115401	6.00E-05	98	4.86E-03
6	3.5	6.946	1.2001	5.788	33.499219	7.00E-05	98	4.76E-03
6.5	3.5	7.382	1.3001	5.678	32.239965	7.00E-05	98	4.06E-03
7	3.5	7.826	1.4001	5.59	31.243636	8.00E-05	98	4.00E-03
8	3.5	8.732	1.5001	5.821	33.883404	7.00E-05	98	3.05E-03
8.5	3.5	9.192	1.6001	5.745	33.0009	7.00E-05	98	2.68E-03
9	3.5	9.657	1.7001	5.68	32.265287	7.00E-05	98	2.37E-03

9.5	3.5	10.12	1.8001	5.624	31.630861	8.00E-05	98	2.42E-03
10	3.5	10.6	1.9001	5.576	31.092025	8.00E-05	98	2.17E-03
10.5	3.5	11.07	2.0001	5.535	30.633162	8.00E-05	98	1.96E-03
11	3.5	11.54	2.10001	5.495	30.197354	8.00E-05	98	1.78E-03
11.5	3.5	12.02	2.20001	5.464	29.851051	8.00E-05	98	1.62E-03
12	3.5	12.5	2.30001	5.435	29.536605	8.00E-05	98	1.48E-03
12.5	3.5	12.98	2.40001	5.408	29.249826	8.00E-05	98	1.36E-03
13	3.5	13.46	2.50001	5.384	28.987224	8.00E-05	98	1.25E-03
13.5	3.5	13.95	2.60001	5.365	28.787131	8.00E-05	98	1.16E-03
14	3.5	14.43	2.70001	5.344	28.562875	8.00E-05	98	1.08E-03
15	3.5	15.4	2.80001	5.5	30.249784	8.00E-05	98	1.00E-03
15.5	3.5	`15.89	2.90001	5.48	30.0304	8.00E-05	98	9.32E-04
16	3.5	16.38	3.0001	5.46	29.8116	8.00E-05	98	8.71E-04

The following figures shows the stress analysis using Bousssinesq theory for Merewa site

x	y	r	z	r/z	(r/z)2		Q(kN)		merewa
0	3.5	3.5	0.0001	35000	1.225E+09	9.00E-24	98	8.82E-14	
0.5	3.5	3.535	0.1001	35.31	1247.127	9.00E-09	98	8.80E-05	
1	3.5	3.64	0.2001	18.19	330.90901	2.00E-07	98	4.90E-04	
1.5	3.5	3.808	0.3001	12.69	161.01335	1.00E-06	98	1.09E-03	
2	3.5	4.031	0.4001	10.07	101.50525	5.00E-06	98	3.06E-03	
2.5	3.5	4.301	0.5001	8.6	73.964815	1.00E-05	98	3.92E-03	
3	3.5	4.61	0.6001	7.682	59.013938	2.00E-02	98	5.44E+00	
3.5	3.5	4.95	0.7001	7.07	49.990818	3.00E-05	98	6.00E-03	
4	3.5	5.315	0.8001	6.643	44.128381	3.00E-05	98	4.59E-03	
4.5	3.5	5.701	0.9001	6.334	40.116271	4.00E-05	98	4.84E-03	
5	3.5	6.103	1.0001	6.102	37.239161	5.00E-05	98	4.90E-03	
5.5	3.5	6.519	1.1001	5.926	35.115401	6.00E-05	98	4.86E-03	
6	3.5	6.946	1.2001	5.788	33.499219	7.00E-05	98	4.76E-03	

6.5	3.5	7.382	1.3001	5.678	32.239965	7.00E-05	98	4.06E-03	
7	3.5	7.826	1.4001	5.59	31.243636	8.00E-05	98	4.00E-03	
8	3.5	8.732	1.5001	5.821	33.883404	7.00E-05	98	3.05E-03	
8.5	3.5	9.192	1.6001	5.745	33.0009	7.00E-05	98	2.68E-03	
9	3.5	9.657	1.7001	5.68	32.265287	7.00E-05	98	2.37E-03	
9.5	3.5	10.12	1.8001	5.624	31.630861	8.00E-05	98	2.42E-03	
10	3.5	10.6	1.9001	5.576	31.092025	8.00E-05	98	2.17E-03	
10.5	3.5	11.07	2.0001	5.535	30.633162	8.00E-05	98	1.96E-03	
11	3.5	11.54	2.10001	5.495	30.197354	8.00E-05	98	1.78E-03	
11.5	3.5	12.02	2.20001	5.464	29.851051	8.00E-05	98	1.62E-03	
12	3.5	12.5	2.30001	5.435	29.536605	8.00E-05	98	1.48E-03	
12.5	3.5	12.98	2.40001	5.408	29.249826	8.00E-05	98	1.36E-03	
13	3.5	13.46	2.50001	5.384	28.987224	8.00E-05	98	1.25E-03	
13.5	3.5	13.95	2.60001	5.365	28.787131	8.00E-05	98	1.16E-03	
14	3.5	14.43	2.70001	5.344	28.562875	8.00E-05	98	1.08E-03	
15	3.5	15.4	2.80001	5.5	30.249784	8.00E-05	98	1.00E-03	
15.5	3.5	`15.89	2.90001	5.48	30.0304	8.00E-05	98	9.32E-04	
16	3.5	16.38	3.0001	5.46	29.8116	8.00E-05	98	8.71E-04	

The following figures shows the stress analysis using Bousssinesq theory for Jiren site

x	**y**	**r**	**z**	**r/z**	**0.478**	**(r/z)2**	**(1 +(r/z)2)5/2**	**I δ**	**Q(kN)**	δz(kN/m^2
0	3.5	3.5	0.0001	35000	0.478	1E+09	3.00E+22	1.59E-23	66.5	1.06E-13
0.5	3.5	3.535	0.1001	35.31	0.478	1247.1	6.00E+07	7.97E-09	66.5	5.29E-05
1	3.5	3.64	0.2001	18.19	0.478	330.91	2.00E+06	2.39E-07	66.5	3.97E-04
1.5	3.5	3.808	0.3001	12.69	0.478	161.01	3.00E+05	1.59E-06	66.5	1.18E-03
2	3.5	4.031	0.4001	10.07	0.478	101.51	1.00E+05	4.78E-06	66.5	1.99E-03
2.5	3.5	4.301	0.5001	8.6	0.478	73.965	5.00E+04	9.56E-06	66.5	2.54E-03

3	3.5	4.61	0.6001	7.682	0.478	59.014	3.00E+04	1.59E-05	66.5	2.94E-03
3.5	3.5	4.95	0.7001	7.07	0.478	49.991	2.00E+04	2.39E-05	66.5	3.24E-03
4	3.5	5.315	0.8001	6.643	0.478	44.128	1.00E+04	4.78E-05	66.5	4.97E-03
4.5	3.5	5.701	0.9001	6.334	0.478	40.116	1.00E+04	4.78E-05	66.5	3.92E-03
5	3.5	6.103	1.0001	6.102	0.478	37.239	9.00E+03	5.31E-05	66.5	3.53E-03
5.5	3.5	6.519	1.1001	5.926	0.478	35.115	8.00E+03	5.98E-05	66.5	3.28E-03
6	3.5	6.946	1.2001	5.788	0.478	33.499	7.00E+03	6.83E-05	66.5	3.15E-03
6.5	3.5	7.382	1.3001	5.678	0.478	32.24	6.00E+03	7.97E-05	66.5	3.13E-03
7	3.5	7.826	1.4001	5.59	0.478	31.244	6.00E+03	7.97E-05	66.5	2.70E-03
8	3.5	8.732	1.5001	5.821	0.478	33.883	7.00E+03	6.83E-05	66.5	2.02E-03
8.5	3.5	9.192	1.6001	5.745	0.478	33.001	7.00E+03	6.83E-05	66.5	1.77E-03
9	3.5	9.657	1.7001	5.68	0.478	32.265	6.00E+03	7.97E-05	66.5	1.83E-03
9.5	3.5	10.12	1.8001	5.624	0.478	31.631	6.00E+03	7.97E-05	66.5	1.63E-03
10	3.5	10.6	1.9001	5.576	0.478	31.092	6.00E+03	7.97E-05	66.5	1.47E-03
10.5	3.5	11.07	2.0001	5.535	0.478	30.633	6.00E+03	7.97E-05	66.5	1.32E-03
11	3.5	11.54	2.10001	5.495	0.478	30.197	5.00E+03	9.56E-05	66.5	1.44E-03
11.5	3.5	12.02	2.20001	5.464	0.478	29.851	5.00E+03	9.56E-05	66.5	1.31E-03
12	3.5	12.5	2.30001	5.435	0.478	29.537	5.00E+03	9.56E-05	66.5	1.20E-03
12.5	3.5	12.98	2.40001	5.408	0.478	29.25	5.00E+03	9.56E-05	66.5	1.10E-03
13	3.5	13.46	2.50001	5.384	0.478	28.987	5.00E+03	9.56E-05	66.5	1.02E-03
13.5	3.5	13.95	2.60001	5.365	0.478	28.787	5.00E+03	9.56E-05	66.5	9.40E-04
14	3.5	14.43	2.70001	5.344	0.478	28.563	5.00E+03	9.56E-05	66.5	8.72E-04
15	3.5	15.4	2.80001	5.5	0.478	30.25	5.00E+03	9.56E-05	66.5	8.11E-04
15.5	3.5	`15.89	2.90001	5.48	0.478	30.03	5.00E+03	9.56E-05	66.5	7.56E-04
16	3.5	16.38	3.0001	5.46	0.478	29.812	5.00E+03	9.56E-05	66.5	7.06E-04