University of Southern Queensland

Faculty of Engineering and Surveying

Soil Pore Blockage as Influenced by Livestock Effluent with Specific Focus on Colloids

A dissertation submitted by

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In fulfilment of the requirements of

Bachelor of Environmental Engineering (Honours)

October 2015

Abstract

Management of livestock effluent, is becoming a mounting concern for beef feedlot operators as they seek to comply with environmental regulation that limits the seepage from beneath effluent storage ponds to a maximum hydraulic conductivity of 1.0×10^{-9} m/s. The literature suggests, particularly work by Bennett & Warren (2015), that this rate is achievable through the compaction of *in-situ* soils to 98% of their maximum dry density. However, there is a lack of research into the effect that liner depth has on colloidal blockage, and thus the need for this research topic.

Using various scales of liner depth to ponded head, all having the same hydraulic gradient, the hydraulic conductivity was tested to determine whether there was any difference between the scales over time. It was found that there was no significant difference and therefore liner depth is of importance to colloidal blockage.

It was made apparent by additional research that colloidal blockage forms a two layer system over time, where the upper layer remains saturated and the layer beneath transitions into unsaturated flow.

This finding adds to the literature regarding soil pore blockage via colloidal entrainment mechanisms, and also adds to the knowledge available to feedlot operators. This research supports the employment of the current guidelines of compacted liner construction, which stress the importance of liner thickness.

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Timothy Colin George 0061032761

Acknowledgments

Firstly I would like to thank my family for helping me out immensely with the construction phase of the project (the biggest and most stressful time of my time at university). Pop your know-how with building stuff was of irreplaceable value to me, and I've surely learned far more than this dissertation has time to detail from my time working with you. Mum thank you for your help with getting my head around some of the scientific writing techniques, and also for always being willing to roll up your sleeves and do some hard labour even if the reward was only three enchiladas, or was it quesadillas?

To Jemima, thank you too for the innumerable times (I'm sure somewhere in that beautiful head of yours you've got the exact number of times) you've driven up to Toowoomba with me to help out with the project, and for being so patient with me.

For the one and only "Big Stirl" for being a fair dinkum bloke and always offering to lend a hand. You have made it into the archives as having said such things as "rrrrrrn woooooah she's got a mind of her own…… wooooah watch out".

Last but certainly not least, I'd like to thank my supervisor Dr. John McL. Bennett for assisting me with the technical side of the project, in which he provided a wealth of knowledge at the cost of a late night email or two.

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1 Chapter 1: Introduction

1.1 Project Overview

Recent research by Bennett and Warren (2015) determined that leaching soils compacted to 98% of their maximum dry density with filtered livestock effluent, resulted in reduction of seepage to below the regulation limit. This was primarily attributed to the entrainment of suspended colloids. This project seeks to directly follow-on and further this research, by investigating the role that liner thickness plays in the entrainment of colloids.
With increased liner thickness and a proportionally increased pressure head, it is expected that the extra tortuosity and thus higher chance of colloids adhering to the soil pore network will result in liner thickness being of significance. However if the opposite is true, that being that liner thickness is not significant, it would potentially result in a cost reduction of installing livestock effluent ponds.

1.2 Project Objectives

To further the research conducted by Bennett and Warren (2015), this research project seeks to establish supporting evidence to the study's discoveries in relation to the role of livestock effluent suspended particulate in sealing ponds. Specifically the ability for it to reduce beneath pond seepage to the regulation limit of $1.0\text{x}10^{-9}$ m/s (Skerman 2000).

Furthermore, it seeks to investigate the effect of pond liner depth on increasing colloidal adhesion via maintaining a constant hydraulic gradient, but reducing the scale of the system. This is of particular importance because it challenges the fundamental equations that are used to describe the physics of hydraulic conductivity, namely 'filtercake theory' and the Darcy equation for hydraulic conductivity.

We aimed to achieve these goals through the construction of an apparatus, to facilitate the leaching of filtered effluent through soil compacted to 98% of its maximum dry density. The apparatus will allow for the replication of said process, for three differing scales of the "best practice" recommendation of a 450 mm liner for a pond depth of 2500 mm.

Following this the incremental discharge measurement of leachate beneath the soil liner over time allowed for the collation of results. This enabled the calculation of the seepage rate change with time, and consequently allowed for comparison of this between the scaled replicates , and also the results of Bennett and Warren (2015)

From the analysis of the results obtained from the comparison it may be possible to confirm that adhesion creates sufficient hydraulic decline in soils with coarse and fine textures, and also whether liner depth is significant.

To support the finding of whether liner depth is significant or not via the comparison of seepage rate decline, an investigation into the depth of colloid entrainment within the soil liner was conducted. In furtherance to this, an investigation into the efficiency of colloid entrainment through analysis of outflowing effluent, compared with that of the inflowing fluid occurred.

2 Chapter 2: Review of the relevant literature

The global population has been rapidly increasing since the industrial revolution leading to ever increasing strain on primary industries to provide them with sustenance (Godfray et al. 2010). Furthermore, the ever decreasing finite amount of arable land and the difficulty of remaining economically competitive, have led to large-scale production intensification (Thompson 1972) . The beef industry, in particular, has felt a significant shift from conventional grazing to more intensive methods of rearing (AFLA/MLA 2014).

In contrast to the benefits of this intensification, there are also negative effects, namely intensification of accumulated effluent. This poses significant risk to the immediate environment, and hence, appropriate control measures need to be installed for the purpose of pollution mitigation. Of particular concern is risk to groundwater resources. The current widely accepted method of mitigation is to capture livestock runoff in effluent ponds so that may be concentrated further through evaporation (Hills & Kemmerle 1981). Seepage from these storage ponds requires close management, with governing bodies within Australia regulating the coefficient of permeability (saturated hydraulic conductivity) for the lining of contaminated waste storage facilities to a maximum of 1.0×10^{-9} m/s or 31.5 mm/year (Skerman 2000).

Current methods of reducing seepage beneath effluent ponds, include; installing polymer membranes to line the in-situ soil (Ortego et al. 1995); importing clay to serve as a compacted liner (Maheshwari & Turner 1986); and compacting *in situ* soil at a rate of up to 98% of the maximum dry density. Although the first two methods are functional, they are expensive relative to the third method and may significantly impact the viability of a business. As a result there is an industry requirement for research into treatment of *in-situ* soil to identify where it is feasible for use against regulatory guidelines. It has been suggested by Mohamed and Antia (1998) that a maximum of 1.0×10^{-9} m/s beneath pond seepage can be achieved by compacting the in-situ clay liner.

2.1 Soil hydraulic conductivity

When analysing the rate of infiltration through a permeable medium, a crucial concept to understand is that of Hydraulic Conductivity (HC). The analysis behind the determination of whether liner thickness has a notable impact on seepage reduction, therefore requires rudimentary understanding of HC before being able to comprehensively assess the impact of the relevant factors affecting it.

Hydraulic Conductivity, with relevance to the soil matrix is defined as the parameter by which a volume of soil has the ability to conduct liquid inside its volume. In the saturated state (i.e. all voids within the soil bulk density are filled with solution), the direction of flow is the same as that of gravity which is the presiding potential (Hillel 2004; Smith 2000). Saturated flow occurs under the influence of a gradient hydraulic pressure potential, or head, consisting of the atmospheric pressure and hydrostatic pressure under natural conditions (Dirksen 1999; Hillel 2004; Smith 2000). When measuring flow through liners of ponds, it is assumed that the flow is saturated. Thus, in such cases, HC can be determined empirically through the use of the law, Equation 2.1 developed by Darcy (1856) for application to one-dimensional problems. Original apparatus set-up is shown in Figure 2.1

$$K\left(\frac{m}{s}\right) = q\left(\frac{m}{s}\right)\frac{L(m)}{\Delta H(m)} = \frac{V(m^3)\;L(m)}{A(m^2)\;t(s)\;\Delta H(m)}$$

Equation 2.1: Darcy's Law for hydraulic conductivity (Darcy, 1856)

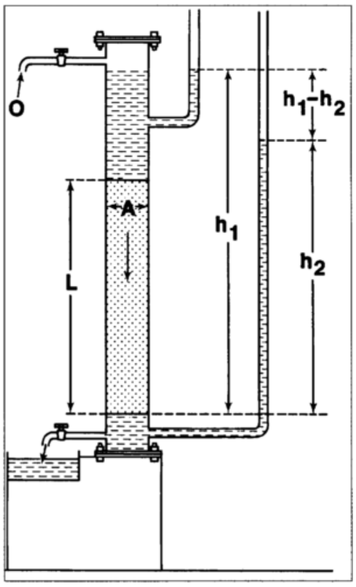

Figure 2.1: Apparatus used by Darcy to develop Darcy's law (Darcy, 1856)

Where, *q* is the flux or specific flow rate measured as the flow rate per unit cross-sectional area which is, volume, *V*, per unit time, *t*, per unit cross-sectional area, *A*; *K* is the hydraulic conductivity; *L* is the thickness of the porous media; *ΔH* is the hydrostatic head measured from the base of the saturated porous media.

The application of Darcy's Law functions on the principle that the subject flow is laminar, as this results in the average flow velocity being proportionate to the hydraulic pressure gradient (Hillel 2004; Smith 2000). Saturated flow in soils is typically laminar as the flow path conditions coincide with those often associated with induced laminar flow: narrow and tortuous causing low velocities. The aforementioned proportionality results from the frictional forces caused by shear resistance at the circumference of the flow path, being equal to the pressure gradient required for inducing fluid movement. Increase in the flow path diametre and/or flow velocity results in the proportionality no longer being valid and the flow condition becoming turbulent. However, in saturated soils of where soil has been compacted to have high bulk density larger flow paths are considered to be uncommon, due to micro-pores (pores of small diameter resulting in very narrow flow paths) presiding. Any macro-pores present prior to compaction are reduced in diameter significantly resulting in a soil medium with a high percentage of micro and meso pores In saturated compacted clay liners, this assumption would be especially applicable: thus Darcy's law is suitable for this research.

Measurement of hydraulic conductivity allows for judgement of the characteristics governing flow such as the features of the soil (pore size proportions and distributions, tortuosity of the flow path, chemical characteristics) and of the permeating solution (viscosity, density, chemical characteristics) (Sumner 1993). Features become apparent due to the change in HC as a result of soil/solution interaction. Detrimental interactive effects can occur due to the chemical composition and concentration of soil solution and soil resulting in degradation of the soil matrix structure through the changes to swelling and dispersive-potential (Ezlit et al. 2013; Frenkel, Goertzen & Rhoades 1978)

Soil structure, after chemically and/or mechanically induced change, as well as in its original form is major factor affecting the rate of HC (Assouline, Tessier & Tavares-Filho 1997; Frenkel, Goertzen & Rhoades 1978). Further blockage can occur due to deposition of organic matter upon the soil surface (through the straining mechanism, possible biological growth), or within the soil matrix (via the filtration mechanism, or possible biological growth) (Meyer, Olson & Baier 1972; Tanner, Sukias & Upsdell 1998).

These final two mechanisms are integral to further research into the potential for suspended organic particulate to decelerate HC in compacted *in-situ* soil liners beneath effluent ponds. However, the remainder of this literature review will look at the proportional effect that all contributing factors have on decelerating HC, which include the various chemical, physical, and biological effects.

2.2 Factors controlling Hydraulic Conductivity within the soil matrix

2.2.1 Clay content and relative pore size

Soil texture and resulting structure is fundamental in the formation of pore networks which are essential for infiltration to occur. Classification of a soil includes a description of its texture, which is essentially a description of the relative gravimetric proportions of the primary soil constituents clay, silt and sand (Clay<2 µm, 2 µm<Silt<20µm, 20 µm<Sand<2000 µm). Isbell (2002) describes increased resistance to flow found in soils with a high proportion of clay particles (fine textured e.g. clayey soils), as such soils have small pore spaces and hence higher frictional resistance compared to soils with high proportions of sand particles (coarse textured e.g. sandy soils).

Soils with a higher proportion of micro/meso-pores (i.e. fine textured soils) have a higher water retention capacity due to increased matric potential or suction (Saxton et al. 1986). However, in saturated flow suction forces are non-existent (i.e. matric potential equals zero) and hence do not affect the flow (Hillel 2004).

2.2.2 Clay swelling properties

Upon wetting, some clays swell in volume, which significantly reduces the HC of a soil because it impedes on the extent of pore networks, and pore diameter (Blackman et al. 2008; Frenkel, Goertzen & Rhoades 1978), severely limiting macroscopic flow. This swelling has been attributed to the crystalline structure and chemical composition of the clay minerals. Expanding silicate clay minerals are considered to be of layer silicate structure, composed of sheets of interlocking crystal units of either octahedral Aluminium-Magnesium (central Al^{3+} or Mg^{2+} atom surrounded by six $O^{2-}$and/or OH^{-} atoms) or tetrahedral Silica (central Si^{4+} surrounded by four O^{2-}) in a 2:1 (trimorphic) sheet ratio (Hillel 2004). The most predominant of these is of the Smectite group, specifically Montmorillonite (Blackman et al. 2008).

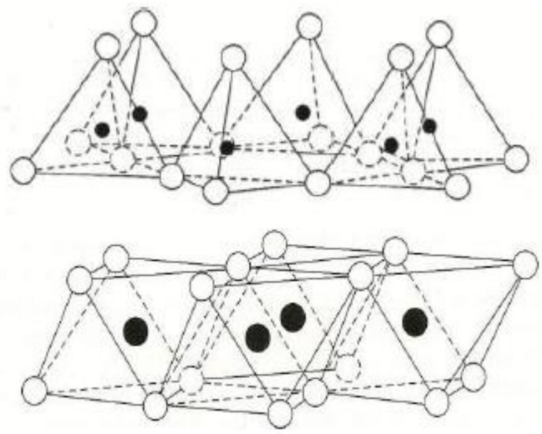

Figure 2.2: (top to bottom) Tetrahedral Silica sheet, Octahedral Aluminium/Magnesium sheet (Hillel 2004)

Expanding trimorphic ratio clays have a central tetrahedral sheet bonded parallel on both sides by two octahedral sheets. This results in the exposed faces only exhibiting weak oxygen-oxygen, and oxygen-cation linkages to the other composite layers resulting in loose stacking (Brady 2008). The spaces created by this loose stacking attract exchangeable cations and accompanying water molecules into themselves. Alongside the higher attraction caused by higher specific surface area, Smectite group minerals also have high negative charges due to atoms of similar size but lower valency being isomorphically substituted for the original central atom at a higher rate than dimorphic (1:1 ratio) minerals (Brady 2008; Evangelou & Phillips 2005). Si^{+4} in the tetrahedral crystal can be replaced with Al^{3+} or Fe^{3+}, whereas the Al^{3+} in the octahedral crystal can be substituted for Mg^{2+}, Zn^{2+} or Fe^{2+} (Rengasamy & Olsson 1993a). The resultant overall more negative charge causes a significant increase in the rate at which cations and associated water are drawn into the structure (Evangelou & Phillips 2005). Furthermore, stacks of these layers tend to mass upon one-another creating ultramicropores (0.0001-0.005mm diametre) causing further water retention and extreme swelling (Blackman et al. 2008; Hillel 2004).

Many Australian soils contain smectite to some extent, but most prominently Vertosols have high shrink/swell potential. Hence it is possible that in-situ construction of effluent retention ponds will benefit from the reduction in HC caused by liners of this soil classification (Isbell 2002). Contrarily, the consequent shrinking when this soil is dried, causes deep fissures to form within the structure, causing preferential flow of high rate application water up until the point where increased moisture content causes fissures to be filled with the increased volume of the swollen soil (Chen, Roseberg & Selker 2002).

2.2.3 Clay dispersive/flocculation potential

Awedat et al. (2012b) showed that dispersed clay within a permeating fluid has the ability to cause reduction in HC through entrainment of colloids within the soil pore-network. Dispersion of clay is defined as the separation of individual clay particles from larger soil aggregates (Blackman et al. 2008). The potential for soils to disperse is dependent on the extent of mechanical disturbance the soil has undergone, the amount of sodium ions within the soils cation exchange capacity (CEC) and it's interaction with the electrolyte concentration of the soil solution (Frenkel, Goertzen & Rhoades 1978; Quirk & Schofield 1955). Chemical instability induced dispersion is the dominant mechanism by which dispersion occurs, usually through an excess of sodium ions within the soil solution (Evangelou & Phillips 2005). A measure of this excess is called the Sodicity of the soil. Sodic soils in Australia have been found to have an exchangeable sodium percentage (ESP) of >6% (Northcote & Skene 1972). More recent work has shown that Potassium also has an influence on soil dispersion (Bennett, Marchuk and Marchuk Submitted; Rengasamy and Marchuk 2011; Marchuk et al. 2014), which is a concern as many wash down facilities use Potassium based products to replace Sodium based ones.

Dispersed clay is a concern because of its physical and chemical features: it is a negatively charged colloid. Soil colloids, both organic and inorganic, are the active component of the soil solid phase, due to their extremely small size (<0.002 mm). This causes their surface area to be many times greater than their volume, and as such causes them to remain in suspension indefinitely. Infiltration of colloid containing solution into a soil pore network may result in removal of the colloids from the solution (Evangelou & Phillips 2005). In cases where the soil has a fine pore network with a low pore size and high tortuosity – under natural condition, or through artificial increase of the bulk density – colloids become entrained within the network due to physical incompatibility. More prevalent however, is the adsorption of colloids to the soil due to charge interaction(Brady 2008) .

2.2.4 Diffuse double layer theory

The diffuse double or electrostatic layer (DDL) is the chemical principle which governs dispersive potential of aggregates into their component parts (Marshall 1988; McLaren 1996). Soil colloids are predominantly negatively charged, but through the use of Coloumbic/electrostatic forces they neutralise themselves by attracting cations onto their exposed surfaces. These cations are normally within the soil solution, however when dry they are held closely to the colloids. When wet the ions are liberated into the soil solution to an extent: they are still attracted to the colloids and form a relatively high concentrated layer of cations around the colloids surface with concentration reducing with increased offset from the colloid surface due to differing valencies (Evangelou & Phillips 2005; Marshall 1988; McLaren 1996). This layer of increased concentration relative to the surrounding solution is known as the diffuse double layer. The dissociation from the surface occurs because of osmotic forces which oppose the columbic forces of attraction. Osmotic forces seek to diffuse the solutes so that the soil solution reaches equilibrium concentration (Blackman et al. 2008).

The thickness of the layer is manipulated by the valencies of ions contained within the solution. Higher charge causes attraction to be stronger and hence the thickness of the double diffuse layer to be smaller (Hillel 2004). Cation exchange reactions can affect clay dispersion as a result of the valency of ions and concentration of the soil solution. With changes to the concentration of the soil solution cation exchanges occur rapidly. In order to reach equilibrium with the soil solution the composition of the cation exchange capacity adapts, this adaption affects the potential for swelling and dispersion (Evangelou & Phillips 2005; Hillel 2004; Marshall 1988; McLaren 1996). Primarily increases in DDL thickness causes overlap of with that of the neighbouring particle. This overlap means that the negatively charged colloids repulse one another causing aggregate dispersion (Essington 2004).

Cations harbour a hydrated radius which further contributes to the potential for dispersion. Cations with thicker hydrated radii are held less strongly to the colloid surface. The thickness of the hydrated radius of the cation is inversely proportional to its valency i.e. monovalent cations have a larger radius than divalent cations, thus further decreasing the strength with which they are held to the colloid. As a result, the combination causes increased destabilisation of the colloid resulting in dispersion (Evangelou & Phillips 2005). Thus sodic soils are prone to dispersion, as sodium is monovalent and has a thick hydrated radius. Stabilising properties of calcium and magnesium cations are as a result of their higher valencies (Ca^{2+} and Mg^{2+}), with calcium providing the greatest stabilising effect due to its smaller hydrated radius (Sposito 1989).

Addition of these stabilising cations can be achieved through concentration within the soil solution, or electrolyte concentration.

2.2.5 Threshold electrolyte concentration

Despite the presence of high exchangeable sodium concentration (ESP) within the soil solution, to a significant extent the degree of dispersion and swelling may be lessened if the percolating solution has a greater electrolyte concentration (EC) than that of soil solution. This is due to the induced osmotic effect that occurs, resulting in water being expelled from the DDL in favour of attaining equilibrium concentration with the soil water. This results in a contraction of the size of DDL allowing the Coloumbic attraction forces of the colloids to triumph, and as a result soil stability is largely maintained (Blackman et al. 2008; Northcote & Skene 1972).

Studies have documented the effects on permeability reduction due to dispersion and swelling, with many proposing that these mechanisms are the foremost contributors (Quirk & Schofield 1955; Shainberg et al. 1992; Sumner 1993).

SAR (ESP) and electrolyte concentration (EC) play integral parts in the level of permeability that a soil possesses. Coupling low EC, with a high SAR has been shown to significantly reduce the hydraulic conductivity of the soil (Bodman & Fireman 1950). However as stated above, through the use of a percolating solution with a higher electrolyte concentration than that of the soil solution, the soil may remain largely immune to swelling and/or dispersion. Electrolyte concentration is defined as the concentration of metal atom ions within the solution. These ions are initially generated from the breakdown of ionic bonds within salts when dissolved in the presence of the solution (Blackman et al. 2008).

The minimum relative concentration of EC that is required to maintain stability – namely limiting reduction of permeability to 10 to 15% - is known as the Threshold Electrolyte Concentration (TEC), and was first termed as such by Quirk and Schofield (1955). In the same study, it was found that the percolation of soils with solutions of differing metal ions resulted in the determination that reduction of HC was limited the most when leached with 0.1 M $CaCl_2$ generated TEC, compared with a high reduction seen in solutions with 0.25 M NaCl generated TEC.

Furthermore it was shown by Frenkel, Goertzen and Rhoades (1978), and later again by Shainberg et al. (1992) that bulk density, clay content, and organic matter concentration are further factors that influence the magnitude of the TEC. Consequently, a recent study by Bennett and Raine (2012) defined the matter of the extent of TEC to be highly soil specific.

2.2.6 Effect of dispersion on pore networks

Clay dispersion also causes pore blockage through development of a surface sealing layer and/or the entrainment of suspended clay colloids in the soil pore network. Vinten and Nye (1985) suggest that this mechanism depends on the relative size of the colloid to the pore diameter through which it travels. Furthermore Awedat et al. (2012b) found that reductions in HC could be attributed to clay particle entrainment in soils of high bulk density due to smaller diameter pore networks causing clay particles to become entrained from the percolating solution. Figure 2.3 depicts mechanisms by which dispersed colloids cause pore blockage.

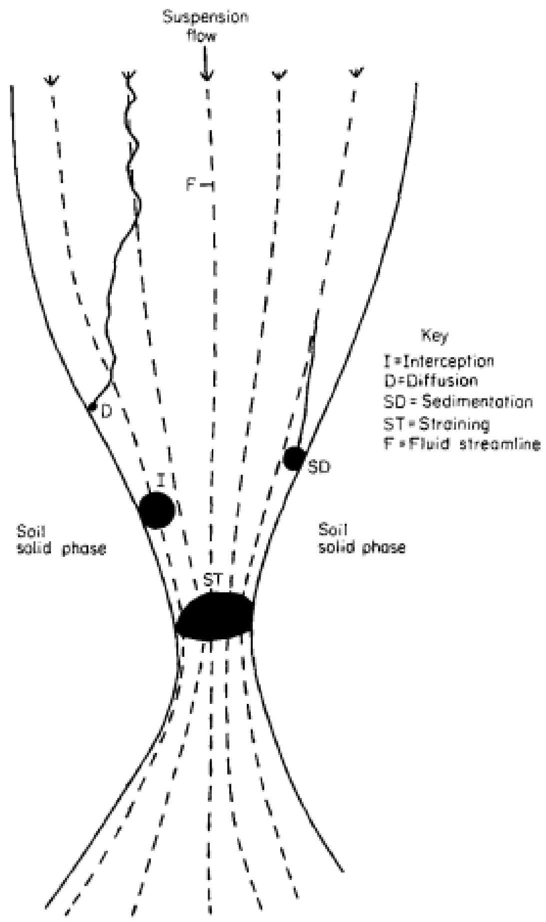

Figure 2.3: Methods of colloidal entrainment (Vinten & Nye 1985)

Colloids can become entrained via two main mechanisms; physical entrainment, whereby the colloid becomes stuck within the pore network tortuosity due to narrowing of pore diametre as seen In Figure 2.3, or were the momentum of the moving colloid causes it to embed into the pore wall in situations where the pore bends abruptly (Vinten & Nye 1985). Alongside this the interaction between the charges of the colloid and the soil pore network can cause them to adhere together (Evangelou & Phillips 2005).

Furthermore it has been found by Rowell, Payne and Ahmad (1969) that with increasing mechanical stress applied to the soil there is a proportional increase in dispersion of clay particles thus swaying the mechanism for blockage from that of swelling, to that dispersed clay colloid entrainment.

2.3 Bulk Density

Density, by definition, is a measure of total mass contained within a volume. With reference to soil applications, “Bulk Density” differs from the prior definition in that it is specifically a measure of the dry soil component and the pore space i.e. neglecting the moisture component (Hillel 2004). Applying mechanical effort to a soil to compact it causes its bulk density to increase as a result. Respectively we often see significant alterations to the soils structure depending on the degree of compaction. Of specific interest to this research is the change to the soils pore network which, as previously indicated, causes changes to the HC of the soil. Thus when attempting to reduce the seepage below effluent ponds through the use of compacted liners, it is important to consider the effect of bulk density.

Volumetric water content has been found to decrease with compaction in soils at high matric potentials, contrarily a slight increase occurs at low matric potentials (Assouline, Tessier & Tavares-Filho 1997). This effect can be attributed to a decrease in the ratio of the larger macro-pores, to the smaller meso-pores. With the application of suction to soils we see the larger pores drain first, and as suction increases there is progressive drainage from the successively larger water filled pore. Thus at a given matric potential all of the meso-pores will contain water whilst all the macro-pores will be empty. By some definitions this is known as the field capacity (FC) of the soil.

Given we understand soil compaction decreases pore diameter, a decrease in hydraulic conductivity should be expected where compaction has occurred. Lin et al. (1996) documented that 90% of total water flux could be attributed to 10% of the active macro-pores (>0.5mm) and meso-pores (0.06-0.5mm). Tarawally et al. (2004) supports this, showing that an increase of meso-pores (<0.5mm) at the detriment of macro-pores (>0.5mm) resulted in a significant decrease in K_{sat}. This is due to the larger adhesion forces associated with meso-pores, as well as an increase in pore tortuosity which leads to the soil having lower wettability (Ferrero et al. 2007; Goebel et al. 2004). In soils analysed by (Zhao et al. 2010), an increase of 8% in bulk density led to a 70% decrease in macro-pores, and a

69% reduction in K_{sat}. Furthermore, Dickerson (1976) found that an increase in bulk density of 20% led to a 68% decrease in macro-pores.

Given that a large reduction in hydraulic conductivity is required when installing clay liners beneath waste storage ponds, a compaction level of 98% of the maximum dry density is considered to be both reliably and economically achievable. To achieve approximate homogeneous levels of compaction throughout the substrate current industrial practices call for multiple layers of 25mm thickness to which compactive effort is applied (Skerman 2000).

Soil moisture content is integral in changing the magnitude of compactive effort applied to achieve maximum bulk density (Kayadelen, Togrol & Sivrikaya 2008). The optimum moisture content (OMC) of a soil is that at which maximum compaction occurs (Hillel 2004). The strength of a soil determines its resistance to compaction: in fine textured soils this is largely due to the cohesion between the smaller particulates, whereas in coarse textured soils the high angle of internal friction is predominantly responsible (Schellart 2000). Due to the lower frequency of meso/micro-pores and much lower specific surface area in coarse textured soil the optimum moisture content is far less than that of fine textured soils. Furthermore, in fine textured soils, moisture causes shear friction between particles (largely colloidal) and reduces electrostatic cohesion forces allowing for greater plasticity of the soil.

Regardless of soil type, increase in bulk density causes reduction in HC, therefore it is a valid mechanism to employ in pond liners in an effort to reduce beneath pond seepage. Also irrespective of soil type is the effect that physical and biological properties of the percolating solution have on reducing HC. However, as the literature suggests, where soils contain a larger proportional colloidal fraction, the surface area is much greater, and the average pore size is much less. Additionally, compactive effort provides greater compaction effect due to cohesion being overcome more easily than the internal angle of friction (coarse soil fragment attribute). On this basis, soils with increasing clay content should provide the most promise for HC reduction in use as pond liners.

2.4 Percolating solution considerations

As previously presented, application of saline-sodic water to soil results in chemical interactions that can affect the physical properties of the soil (Blackman et al. 2008;

Evangelou & Phillips 2005). Of importance, it is prudent to note that salinity (electrolyte concentration) increases soil stability due to compression of the diffuse double layer (osmotic pressures), whilst the presence of monovalent cations with large hydrated radius has the opposite effect (sodium and potassium, to a lesser extent) (Quirk and Schoefield 1955; Ezlit et al 2013; Rengasamy and Marchuk 2011). Thus, excess salinity dominated by sodium contained within the soil solution might cause a soil to swell and disperse, or remain stable depending on the threshold electrolyte concentration of the soil.Awedat et al. (2012b) observed soils to swell and disperse resulting in reduction of HC through constriction and blockage of pores where water content of EC 2 dS/m and SAR 120 applied. However, in the presence of threshold amounts of soluble calcium or magnesium these effects can be somewhat avoided (Rengasamy & Olsson 1993a). Replacement of the Na^+ with Ca^{2+} or Mg^{2+} cations results in lowered dispersion and swelling via compression of the DDL as they are divalent in charge meaning one of these ions is required to satisfy the same charge as two Na^+ or K^+ ions. This means that the physical space of the cations is also reduced, even though they have relatively large individual hydrated radii.

Numerous authors have studied the effect of irrigation water containing high Na^+ on clay dominated soil properties (Awedat et al. 2012a; Aydemir & Najjar 2005; Beltrán 1999; Ben-Hur et al. 1985; Bodman & Fireman 1950; Cass & Sumner 1982; Choudhary et al. 2011; Oster & Schroer 1979; Rengasamy & Olsson 1993b) Observed interaction of the highly Na^+ dominated saline solutions with the clay dominated soil concluded that significant swelling and dispersion occurred, resulting in notable reduction of HC up to depths up to 120cm. Reduction in HC was most generally ascribed to dispersed clay particles forming a surface sealing layer, as well as becoming entrained via migration deeper into the soil profile.

Chang et al. (2005) showed that the use of secondary effluent for irrigation applications requires treatment to adjust its high SAR to maintain an appreciable amount of divalent cations so that adverse effects on cropping soil are minimised. However, the aim is to limit seepage under effluent ponds, so in such a case, the presence of solution dispersive properties should result in desirable outcomes where pores are further blocked by dispersed clay. Nevertheless, Bennett and Warren (2015) demonstrated that this should not be a definite expectation due to the threshold electrolyte concentration being soil specific. They used a synthesised solution to matching the EC and SAR of the effluent source applied to two different soils and observed that for one soil the HC marginally increased (considered maintained), whilst for the other soil significant decline was observed. This highlights that dispersive cations may provide a useful reduction, but that this is a function of both solution EC and the soil specific TEC, as proposed by (Bennett & Raine 2012)

2.5 Potential for effluent solutions to seal clay lined ponds

Literature reviewed to this point suggests that interaction between soil compaction (pore diameter reduction), solution dispersive characteristics, and entrainment of small particulate (primarily clay) provides reasonable expectation of significant and rapid reduction in soil HC, which further supports the likelihood of effluent characteristics and *in situ* soil characteristics interacting to produce favourable reductions in pond seepage. Reduction in soil permeability due to the application of effluent solutions has previously been shown to occur (Cihan, Tyner & Wright 2006; Lado, Ben-Hur & Assouline 2005; Meyer, Olson & Baier 1972; Tyner & Lee 2004). Mechanisms suggested to cause HC reduction have been documented as: chemical, biological, and physical pore blockage (de Vries 1972; Rice 1974; Vinten, Mingelgrin & Yaron 1983).

2.5.1 Bio-physical properties

Lado, Ben-Hur and Assouline (2005) tested the effect that a treated sewage effluent solution had on reducing soil HC when used for irrigation practices, finding that compared with fresh water the reduction in HC was significant. Vinten, Mingelgrin and Yaron (1983) attributed much of the reduction in HC to high concentrations of organic suspended solids within effluent solutions causing physical pore blockage. Contrarily, where soil pores are smaller than the organic suspended solids it was found that seepage reduction was due to a physical seal layer forming atop the soil surface, known as a sludge layer (Cihan, Tyner & Wright 2006; Ham 2002; Meyer, Olson & Baier 1972; Tyner & Lee 2004). A long term observation of untreated effluent retention ponds found that reduction of HC after the formation of sludge layer was significant resulting in average seepage rates of 1.8×10^{-9} m/s (Ham 2002).

Additionally after a significant sludge layer had been developed with time it was shown by Meyer, Olson and Baier (1972) that the presence of contaminants within leached water beneath ponds was significantly less than that found in adjacent wells.

The majority of literature points towards a physical seal occurring atop the soil due to straining of particles that are physically larger than the soil pores, resulting in a gradual decrease in soil permeability (Ham 2002; Meyer, Olson & Baier 1972). Furthermore, in soils where repulsive charges exist between solution colloids and the soil body, the straining mechanism may extend beyond simple physical size incompatibility (Bradford et al. 2006). With time, the deposition of particles due to the effects of straining has been found to form a sludge layer, which has an overwhelming effect on reducing permeability (de Vries 1972; Ham 2002; Tanner, Sukias & Upsdell 1998).

The build-up of strained particulates, known as a sludge layer, has a similar effect on hydraulic conductivity as that considered in filtercake theory, and hence this theory may provide a model that numerically describes the sludge layer (Abboud & Corapcioglu 1993; Tyner & Lee 2004). The application of this model suggests that reduction in hydraulic conductivity is purely due to build-up of a sludge layer, and hence the soil depth is unimportant (Cihan, Tyner & Wright 2006). This is because no suspended particles are considered to be captured within the soil matrix, and hence the permeability of the soil itself remains constant (Abboud & Corapcioglu 1993). The theory manipulates Equation 1.2.1.1 (Darcy's equation) to account for the three phases that occur along the time series: initial stage, where the seepage is dominated by the soil HC because no sludge layer has yet formed and therefore can be defined using Darcy's law (equation 1.2.1.1); transition stage, where the HC of the seal is higher than that of the soil and hence total permeability is governed by both (equation 1.5.1.1); and the late stage, where the HC of the seal is much lower than that of the soil and hence permeability is entirely depended on the seal (equation 1.5.1.2) (Cihan, Tyner & Wright 2006).

$$I(m) = \frac{K_{seal}\left(\frac{m}{s}\right)}{\alpha}\left(-\frac{L_{seal}(m)}{K_{soil}\left(\frac{m}{s}\right)} + \sqrt{\left(\frac{L_{soil}(m)}{K_{soil}\left(\frac{m}{s}\right)}\right)^2 + \frac{2H(m)t(s)}{\frac{K_{seal}\left(\frac{m}{s}\right)}{\alpha}}}\right)$$

Equation 2.2: Cumulative infiltration for K whilst K_{seal}>K_{soil} (Cihan, Tyner & Wright 2006)

$$I(m) = \sqrt{2H(m)t(s)\left(\frac{K_{seal}\left(\frac{m}{s}\right)}{\alpha}\right)}$$

Equation 2.3: Cumulative infiltration for K when K_{seal}<K_{soil} (Cihan, Tyner & Wright 2006)

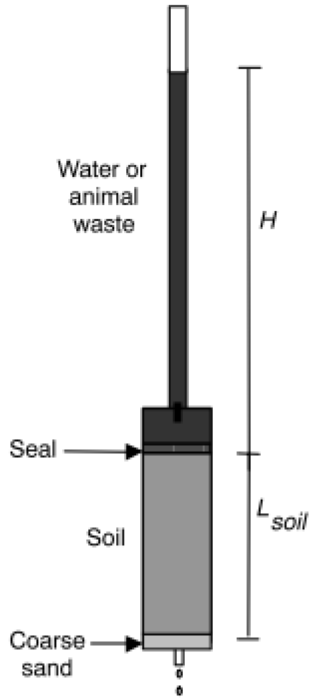

Figure 2.4: example of filtercake experimental apparatus used by Cihan, Tyner and Wright (2006)

Where, I is the cumulative infiltration of the filtrate, K_{seal} and K_{soil} are the HC's for the seal (sludge layer) and soil respectively, L_{seal} and L_{soil} are the thicknesses of the seal and the soil respectively, H is the pressure head applied taken from the top of the soil to the free water surface, t is the time since first application of head to soil, and α is a coefficient relating I to K which is dependent on the concentration of solids within the filtrate and the average pore size within the seal (Cihan, Tyner & Wright 2006).

However, due to the Australian and New Zealand intensive livestock industry employing practices that remove much of the particulate loading of effluent prior to introduction into effluent ponds, the notion of sludge layer build up is somewhat less relevant (MLA, ALFA & FIAC 2012). Therefore it is suggested that the nature of pore blockage following chemical manipulation is largely due to the entrainment through adhesion filtration of smaller inorganic and organic particulates within the below surface soil matrix (Rice 1974).

Further research into reduced-load effluent seepage suggests that additional blockage is caused by the biological growth and the subsequent deposition of residuals by microbes within the soil matrix (Baveye et al. 1995; Rice 1974). The growth of microbes is encouraged with the high nutrient availability contained within effluent.

In the recent past there has been research that has focused entirely on the effect that suspended colloids have on blockage this is discussed further in the next section.

2.5.2 Specific past research

This research project follows on directly from research conducted by Bennett and Warren (2015) who studied the impact that suspended particulate within effluent had on HC of highly compacted clay liners. Two different soils were compacted to 98% of their maximum dry density, and then subjected them to different leachates designed to isolate the effect that suspended particulates. The isolation was conducted by leaching with a chemical simulant and then determining the difference between this and that of filtered effluent. It was discovered that HC could indeed be reduced to below the regulation limit of 1.0×10^{-9} (m/s): dependent on clay content within the soil, reduction was either entirely due to the suspended organic matter within the effluent, or it was a contributing factor alongside the importance of threshold electrolyte conductivity. Thus it is shown that absence of sludge layer inducing particle sizes it is still possible to reduce hydraulic conductivity drastically with time.

Bennett and Warren (2015) extended the experiment runtime from that of previous unreleased research conducted by Bennett, Dalby and Raine (2011)which found that suspended particulate leached through compacted feedlot soils did indeed decrease HC to significantly, with the clay dominated soil achieving a HC below the regulation maximum. In the sandy soil the reduction within the timeframe was not enough to fall below the guideline rate, yet through extrapolation of the trend it was hypothesised that it would, thus the need for the extended timeframe investigation conducted by Bennett and Warren (2015).

It was shown by Awedat et al. (2012b) that the application of suspended clay colloids within leachate to soils of mechanically induced high bulk densities resulted in HC reduction with time, due to entrainment of said clay colloids. This was attributed to physical capture, and also electrostatic adhesion of the particulates against the pore walls thus reducing their diameter. It was further suggested by Awedat et al. (2012b) that blockage due to this primarily occurred within the layers close to the soil surface. Therefore this finding justifies the use of filtercake theory despite the absence of a separate sludge layer, which was also a finding of Bennett and Warren (2015)

Research by Bean, Southcott and Lott (1999) discovered that there was a significant difference in the reduction in HC of compacted soil columns leached with feedlot effluent and those leached with rainwater. Upon bisection of the soil cores it was evident that particulate from the effluent had become entrained within the soil due to an obvious colour stain.

However, the investigation conducted by Bennett and Warren (2015) only considered liner thicknesses of 5 cm with a highly exaggerated hydraulic gradient. This is not representative of ponds in reality, as evident from recommended dimensions provided by *National Guidelines for Beef Cattle Feedlots in Australia* 2012). Due to the small cores used by Bennett and Warren (2015), it was not warranted to investigate the distribution of colloids within the soil pore network, although it is highly likely that they are concentrated near to the surface. Collectively there is a lack of literature regarding the effect that liner thickness has on colloidal entrainment. With regards to investigations into the mechanisms by which bulk density aids colloidal blockage, it is understood that the resultant increase in tortuosity has a significant effect. Therefore we may speculate that this concept applies in the case of increased liner thickness, where a proportional increase in tortuosity also occurs (Hillel 2004). This lends to the concept that liner thickness is significant, however, if the hydraulic head applied is also proportionally increased it could reasonably be expected that the increased energy would negate the effect of tortuosity. Thus there is a novel need for the research conducted in this dissertation.

2.6 Conclusion

The mechanisms that effect HC with regard to being leached with livestock feedlot effluent have been investigated. These include the properties of the soil and effluent that interact, including their chemical, physical, and biological makeup. The focus was largely on the ability for colloidal particles to cause pore blockage in highly compacted soils, with specific emphasis on the study by Bennett and Warren (2015) which determined that this mechanism could reduce hydraulic conductivity to below the guideline rate. The review highlighted the need for research into the effect that liner thickness has with regards to this mechanism of HC reduction.

3 Chapter 3: Methodology

3.1 Overview

The goal of this project is to determine what influence the depth of compacted soil liners has on entrainment of secondary effluent suspended particulate, and thus on hydraulic conductivity (HC). The effect on HC has already been confirmed by Bennett and Warren (2015), yet depth extent of particulate accumulation in the soil liner remains unknown and hence the need for this methodology to discover it.

As discussed in the literature review, it is important to determine what effect pressure head has on the depth to which particulates are deposited. Thus, to experimentally analyse this theory it was decided that variations of the industry recommended ratio of pressure head to liner thickness (hydraulic gradient) was replicated and then replicated again as half and quarter scale. If soil depth is unimportant, then the flux (discharged volume per cross-sectional area per unit of time) should be equal for the three scales according to Darcy's law. However, if depth of liner is important, then the greater liner depth should result in greater reduction of HC, irrespective of pressure head and hydraulic gradient.

Two soils were selected on the basis of their vastly differing clay content; clay from Macalister, west of Dalby in Queensland; and a sandy-loam from Cullendore, east of Warwick in Queensland. The differing clay contents was essential in comparing the HC of a fine and coarse textured soil. Soils were compacted into PVC columns of varying core lengths of 450mm, 225mm, 112mm depending on the required scale of effluent pond simulation. Compaction occurred in layers of 25mm to 98% of the maximum dry density (MDD). Each soil had six replicates at each scale for the filtered effluent solution, and five replicates of the 50mm cores for the Calcium Chloride solution. Two treatment solutions where prepared:

1. Calcium Chloride $CaCl_2$
2. Filtered Feedlot Effluent

Cores for the number 1 solution were leached for 1740 hours (HC measurements taken approximately every three days), cores for number 1 solution were leached for 17 hours (HC measurements taken approximately every half hour). Retention rate of suspended organic particulate found in the filtered feedlot effluent was determined by comparison of periodical total suspended solids (TSS) of the leachate with the percolate prior to leaching.

3.2 Initial Preparation

A significant component of this project was design of the apparatus to undertake effluent pond seepage scale evaluation. Figure 3.1 shows the final construction used for the experimental procedure. However, given the extent of considered design and trial and error process in construction of the apparatus it is of great importance that this be written with sufficient detail to inform future constructions of this nature (Chapter 4). Considerations in the design included maintenance of a constant head, pump cycle mathematics, economic feasibility, load bearing capabilities, logistical processes, post construction function etc. As such design constraints, lessons learned and future considerations are presented as a separate chapter.

Figure 3.1: Complete assembly of effluent leachate apparatus

3.3 Solution Preparation

The same filtered feedlot effluent that was used by Bennett and Warren (2015) was used in this experiment. This allowed for direct comparison of effects. Preparation of the calcium chloride solution was done with reference to the effluent characteristics (Table 3.2).

Table 3.1: Effluent chemistry data (Bennett & Warren 2015)

Analysis	***Unit***	***Effluent Value***
pH		7.6
Electrical Conductivity	µS/cm	5600
Total Hardness	Mg/L $CaCO_3$	426
Total Alkalinity	Mg/L $CaCO_3$	390
Calcium (Ca^{2+})	mg/L	78.6
Sodium (Na^{+})	mg/L	270
Magnesium (Mg^{2+})	mg/L	55.8
Sodium Absorption Ratio		5.7
Mean particle size	µm	3

3.3.1 Calcium Chloride $CaCl_2$

Leaching calcium chloride solution through soil minimises change to its structure and therefore pore network, thus providing the greatest expected hydraulic conductivity for a given electrolyte concentration. As discussed in the literature review, the application of high soluble calcium cations within the leaching solution minimises swelling and dispersion, while the DDL is compressed depending on the EC. Thus, leaching with this solution provided an initial hydraulic conductivity as reference to the soil specific structure, whilst removing the potential variable effect of osmotic forces for the solutions.

As such, the solution was designed to have the same electrical conductive (EC) as the filtered effluent (5.6 dS/m), which is obtained through the addition of 41.216 g of Calcium Chloride di-hydrate ($CaCl_2(H_2O)_2$) salt per 10 L of de-ionised water.

3.3.2 Filtered Effluent

Filtration of effluent was conducted in accordance with the methodology described in Bennett & Warren (2015). Filtration was intended to reduce the mean particle size to <3µm. Attainment of this goal was confirmed through particle size analysis using Malvern Instruments Ltd™ Zetasizer® instrument. The decreased proportion of macro-pores resulting from compacting soil to the extent required, requires understanding of the contribution to physical pore blockage of strained particles (larger than pore diameter thus creating a sludge layer) and filtered particles (allowed to pass through the surface layer and becoming entrained within the soil pore network).

3.4 Soil Core Preparation

After collection from the field, soils were allowed to air-dry for an extended period of time. Once air dried, the soil was crushed using a mechanical apparatus design to supply sufficient force to break soil peds into smaller aggregates without inducing undue force (Figure 3.2). Final aggregate distribution was <2.36 mm diameter, which was ensured by passing a sieve of respective threshold. As the diameter of soil core cross-sectional areas was 87.5 mm to provide feasibility to the experiment, homogenising the soil using this process ensured that undue preferential flow paths were avoided and that experimental construction was uniform, without crushing the soil to an extent that this affected results.

Figure 3.2: Soil Grinder at the DAFF facility Tor St, Toowoomba

3.4.1 Compaction

For compaction to 98% of the maximum dry density (MDD), determination of the optimum moisture content (OMC) and MDD was essential. Conducting a modified proctor test to produce a moisture to compaction curve, the MDD and OMC where determined for both soils (Figure 3.3). This process followed the procedure detailed in Australian Standard 1289.5.3.2. In preparing the soil cores compaction occurred in flights of 25 mm inside the 87.5 mm diameter PVC pipe at a volume of 0.00015 m^3 for each flight. Following this the moisture of the air dried soil was determined gravimetrically, and then extra moisture was added to the weighed soil portions to achieve approximately OMC.

Figure 3.3: Acquiring OMC gravimetrically left, part of the Proctor test apparatus right

Table 3.2: Soil compaction data

Soil	***MDD (Mg/m^3)***	***OMC (%)***	***98% MDD (Mg/m^3)***	***Air Dry MC (%)***	***Volume (m^3)***	***Dry Mass (g)***	***Air Dry Mass (g)***	***Added moisture mass (g)***	***Total mass at OMC (g)***
Clay	1.491	27.95	1.461	11.4	$1.5x10^{-4}$	180.134	203.32	46.70	250.01
Loam	1.988	8.939	1.949	2.735	$1.5x10^{-4}$	292.94	300.96	18.17	319.13

Soil was allowed to equilibrate in air tight containers once the water to achieve OMC had been added. Flights were then added incrementally to the PVC soil cores (held in place using a compaction cylinder) and compacted using a circular steel plate welded perpendicularly to a steel rod which was struck with a hammer. To ensure that the flight of soil was compacted to the right degree, the steel rod was marked incrementally and visually gauged against the top of the soil cylinder (Figure 3.3).

Figure 3.4: Compaction plate and rod

3.4.2 Chemical analysis

Initial chemical properties of the soil for dispersion (both spontaneous and mechanical), electrical conductivity, pH, and exchangeable cation capacity were measured.

3.4.2.1 pH and electrical conductivity test

40 ml of distilled water was added to 8 g of soil and end-over-end shaking for one hour which was then immediately followed by standing for 20 mins, after which the pH and EC of the soil:water suspension was measured using properly calibrated and rinsed probes. For both pH and EC results are reported on a 1:5 soil:water ratio.

3.4.2.2 Exchangeable Cation Capacity

The 1:5 soil:water suspensions created in the previous section were then used to measure the exchangeable cation capacity. To achieve this, the samples were centrifuged at 3000 rpm until all solid particles settled out (Figure 3.5-left). Following this, the supernatant was removed using a syringe and the sediment was filtered and added into tubes for soluble cation testing. To these samples, pH-adjusted ammonia was added to the sediment, and the tubes were then thoroughly shaken until the ammonia solution enveloped all of the soil surface allowing it to replace the bonded cations on the colloid surface thus freeing them. The suspension was again centrifuged at 3000 rpm and a sample taken of the supernatant liquid.

Figure 3.5: (left to right) Centrifuged samples, chemicals used

3.4.2.3 Dispersion (mechanical, and spontaneous)

For the measurement of dispersion, 40 g of soil was added to a measuring cylinder, which then had deionised water carefully added to it so that the combined volume totalled 220 ml. There are two methods of dispersion measurement; spontaneous, which simulates dispersion resulting from minimal physical disturbance; mechanical, which simulates that resulting from large physical disturbance. To measure spontaneous dispersion the cylinder was inverted twice and then left to stand for 4 hours, at which point observations were conducted. Contrarily, for mechanical dispersion the cylinder was inverted twenty times and then it too was left standing for 4 hours before observations were made (Figure 3.6).

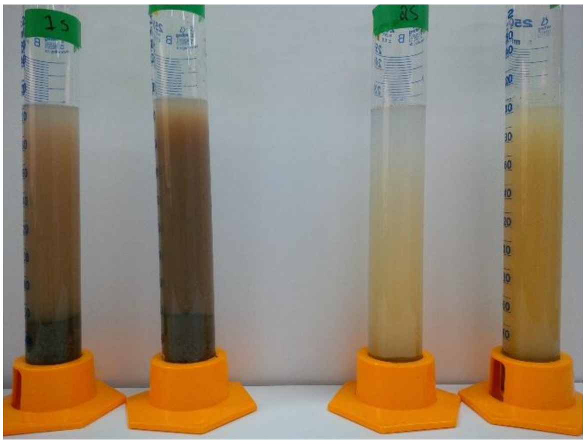

Figure 3.6: Dispersion observation after 4 hours

3.4.2.4 Particle size analysis

For the sake of soil texture identification, a particle size analysis was conducted. To achieve this 50 ml of 10% Calgon and 5 ml of 0.6 M NaOH and de-ionised water to aid the mixing process was added to 40 g of soil. A mixture of the same chemicals without the soil was used as a blank control. All samples were thoroughly shaken for 24 hours. Following this they were added to large measuring cylinders to which de-ionised was also added to bring the volume to a total of 1 L. Following this the sample solutions were thoroughly mixed using a paint mixer and then left to stand for 5 minutes, at which point a hydrometer measurement was taken and compared to that taken from the blank sample. A further measurement was taken after leaving the samples to stand for another 5 hours.

Using the following formulas the percentage of sand silt and clay was determined:

$$\%Silt\ and\ Clay = \frac{(Reading_{5min} - Reading_{Blank})}{mass_{soil}(g)} \times 100\%$$

$$\%Clay = \frac{(Reading_{5hours} - Reading_{Blank})}{mass_{soil}(g)} \times 100\%$$

$$\%Silt = \%Silt\ and\ Clay - \%Clay$$

$$\%Sand = \%100 - \%Silt\ and\ Clay$$

Equation 3.1: Used for sand, silt, clay proportions in particle size analysis

Following this the use of Marshall's diagram was used to determine the texture class.

Table 3.3: Soil texture analysis results

	Gravimetric Proportions (%)			
Soil	***Clay***	***Silt***	***Sand***	***Texture Class***
1	74	7	20	Clay
2	21	3	76	Sandy-Loam

Table 3.4: Soluble cation content of tested soils

	Soluble Cations (meq/L)						
Soil	***Ca^{2+}***	***Mg^{2+}***	***Na^{+}***	***K^{+}***	***SAR***	***EC (μS/cm)***	***pH***
1	0.0033	0.0095	0.1207	0.0091	1.751	0.6105	9.20
2	0.1577	0.1927	5.264	0.0294	12.58	11.03	6.95

Table 3.5: Exchangeable cation content of tested soils

	Exchangeable Cations (cmol/kg)						
Soil	***Ca^{2+}***	***Mg^{2+}***	***Na^{+}***	***K^{+}***	***Ca:Mg***	***ESP (%)***	***CEC (cmol/kg)***
1	0.2051	0.4772	0.0804	0.0831	1.36	9.555	0.8457
2	16.40	19.49	10.10	0.7411	0.583	21.62	46.73

Table 3.6: Spontaneous and mechanical dispersion of tested soils

Soil	***Spontaneous Dispersion***	***Mechanical Dispersion***
1	high – 1980 NTU	very high -2800 NTU
2	low to Medium – 1200 NTU	medium to high – 1650 NTU

3.4.3 Core configuration

Following compaction, the PVC cores had mesh filters attached to the lower portion to prevent downward expansion through possible swelling properties of the soil. An end cap with a 3-mm irrigation tube was attached to the exposed male end of the mesh filter cap (Figure 3.7 left). This was used to redirect leachate into a semi-air-tight container from where it could be collected and weighed (Figure 3.7 right). The container inflow had a small hole to allow for depressurisation, yet avoid evaporation as a significant variable in leachate collection.

Figure 3.7: (left to right) attaching tube to endcap, anti-evaporation system once installed.

3.4.3.1 Effluent Solution columns

The upper portion of the soil core assembly was attached to a standpipe of length respective to the head height required to achieve the desired hydraulic gradient. Measurement of hydraulic conductivity was done through incrementally measuring the mass of leachate produced for a given time period and thus determining the flux. Throughout the experimentation a constant head was applied and as such solving for the HC was undertaken using Darcy’s law.

3.4.3.2 Calcium Chloride Solution columns

These were used to understand the initial condition of the medium, i.e. the maximum possible hydraulic conductivity achievable in the compacted soil samples. As the solution had no particulate within it, there was no need for a specific hydraulic gradient, as long as it was known for the calculation of *K*. The upper portion of the soil core had an end cap attached to it in a similar fashion to that of Figure 3.7, this was then attached to an elevated reservoir whose falling water level during the experimentation was noted (Figure 3.8). The head applied at each incremental measurement was considered to be an average from the start of the time increment to the time the HC measurement was taken. This was then used in Darcy's law for each iteration to produce a HC with reference to the changing head applied. Due to increased frequency of measurements it was considered that evaporation would not pose a significant issue (i.e. was negligible) and therefore no anti-evaporation solution was installed.

Figure 3.8: Calcium Chloride core configuration

3.4.3.3 Initial application of solutions to cores

For the filtered effluent solution a watering can was used to add the first 20cm of fluid to the standpipe in an effort to reduce mechanical disturbance of the soil surface. These cores where not saturated initially. However, the cores with the Calcium Chloride solutions where initially saturated with the same fluid prior to leaching as the aim here was to obtain a reference permeability, rather than simulate an effluent pond.

3.5 Total suspended solids (TSS)

Total suspended solids were determined gravimetrically throughout the leaching time-series in an effort to discover the change in solids retained within the soil. Firstly the TSS of the filtered effluent was determined so that it could be used as a reference for the measurements of the leachate, thus allowing for efficiency of entrainment calculations. Measurements of TSS from leachate samples were taken from a set of standpipes incrementally throughout the experiment runtime. These standpipes were selected at random, although making sure that a sample from one of each variant was taken.

Gravimetric measurement of TSS involves the weighing of a leachate sample in an oven proof container of known weight. This weight is then compared with the oven-dried weight and the percentage of TSS is determined. A preliminary visual comparison was conducted prior to gravimetric measurement (Figure 3.9).

Figure 3.9: Visual comparison of filtered effluent (background) with leachate (foreground)

3.6 Depth of entrained particulate

Through the application of the filtercake theory it was determined that the entrainment of colloids, in line with what was discussed in the Chapter 2, occurring in the first layers of the soil liner up to a certain depth would cause a reduction in hydraulic conductivity for this layer of lower porosity, and consequently the entire layer beneath. However, as the soil beneath this layer of entrained particulate had a resultant relatively higher porosity, it was expected the governing rate of hydraulic conductivity did not fill the pore space of this section entirely and hence the occurrence of unsaturated flow in this section of the soil. Thus to determine the depth at which particulates were entrained, identification of the depth at which the transition from saturated to unsaturated flow occurred was required (Figure 3.10)

To ensure that entrained colloids had reached their capacity, measurement of the depth was conducted once the leaching data had reached a steady state. The column inflows were then closed off, and holes were bored at incremental depths through the PVC casing of the soil cores (Figure 3.10). Increments began at the bottom of the cores and worked upwards, in 30mm, 15mm, and 7.5mm steps for the 1:1, 1:2, and 1:4 scales, respectively. The steps were scaled to make sure that accurate determination of the relative depth could be compared between scales. Borings were done every one minute to allow enough time the potential water to leak from them.

Figure 3.10: Determination of the lower boundary of the saturated zone

4 Chapter 4: Design Methodology

4.1 Overview

In order to be able to measure the hydraulic conductivity seeping through the liner of an effluent pond, an apparatus was constructed to allow simulation of the interaction of ponded effluent and compacted soil liner. As mentioned previously, the key objective of this investigation, being the comparison of the effect liner thickness has on the entrainment of suspended colloids, resulting in provisions being made in the apparatus design to be accommodate for varying scales of liner to ponded effluent head. To reduce the difficulty in the use of Equation 2.1 in the calculation of HC, variables within the equation were designed to remain constant. Thus the apparatus was fitted with a system to maintain constant hydraulic head. Furthermore to protect the collected data from alteration due to evaporation, a system was fitted that reduced this effect almost entirely. The apparatus construction process is presented in further detail below.

4.2 Frame construction

As aforementioned the large number of replicates, detailed in table 4.1, resulted in the need to accommodate the large mass associated with the quantity of liquid and soil. Using the density of the soils detailed in Chapter 3, coupled with assumed density of the liquid (taken to be that of water at 20 degrees Celsius) the mass of the replicates was established as being less than one metric ton. The calculation process is detailed below in Equation 4.1.

Table 4.1: Replicate types and number

Leachate	***Soil***	***Size of Head to Liner***	***Replicates***
Effluent	1	2500mm to 450mm	6
		1250mm to 225mm	6
		625mm to 112.5mm	6
Effluent	2	2500mm to 450mm	6
		1250mm to 225mm	6
		625mm to 112.5mm	6

$$Mass\ of\ replicates = \left(Scale\ 1{:}\,1\ depth\ (m) \ldots .\ Scale\ 1{:}\,nth\ depth\ (m)\right) \times No.\ of\ replicates \times Area\ of\ core\ (m^2) \times density\ (\frac{kg}{m^3})$$

Equation 4.1: Calculation of mass of replicates for use in frame analysis

In addition, the mass of the other apparatus implements – i.e. the plumbing system – needed to be considered. A quick frame material feasibility analysis was conducted, which considered; the economics, both purchasing and assembling the material; the structural stability, whether a frame made of the material would support the required load; the ease of installation of standpipes. Consultation with a tradesperson confirmed the selection of structural 90x40 mm Radiata Pine as the frame material.

To confirm that the frame assembly would definitely support the complex loading, a complex statics analysis of the frame was required. For ease, this analysis was conducted using the frame analysis toolset provided by Autodesk Inventor was utilised (Figure. The program does not support frames made of wood, and therefore a material of the same dimensions was used, with the applied load increased proportionally to the increase in material properties i.e. Poisson’s ratio, Young’s modulus, and shear modulus as well as the tensile, compressive, and shear strength.

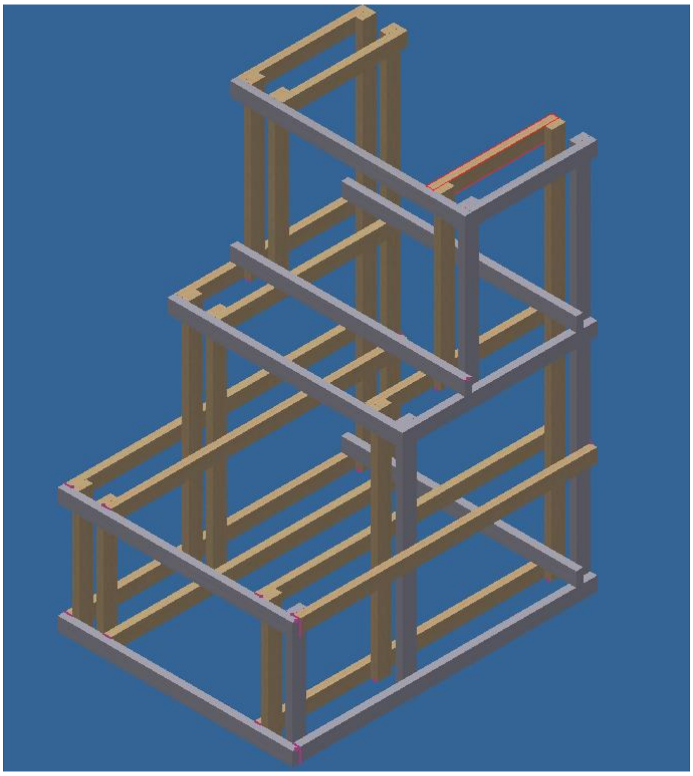

Figure 4.1: The frame after constrains had been put into place in the Inventor software

Dimensions of the frame had to consider; the dimensions of the standpipes; the floor space and roof height of the area allocated; the installation logistics of the plumbing system; and finally the ergonomics of the measurement taking process.

The design also incorporated the ability for sections to be assembled off site and then transported with ease to the location of desired experimentation. For prevention of torsion, members were joined using diagonally spaced wood screws. Following the final assembly at the desired location, the inside back left column was braced against structural column of the existing shelter structure to lend extra structural support in the x and z planes.

4.3 Plumbing system

Inherently the application of head to the soil core requires ponding of leachate on top of the soil. From failings in the design method used by Bennett and Warren (2015), where leaks were a large issue due to the multiple plumbing joins, and reservoirs had to be refilled regularly, it was decided that a method of maintaining constant head was required. Further reasons include; ensuring parameter consistency, meaning later ease of mathematical analysis; add an element of automation so that the experiment could run relatively unsupervised.

To remain cost effective, standard 90mm storm water PVC was used to contain the ponded head for each replicate (Figure 4.1). To ensure firm attachment to the apparatus, the PVC was mounted in place at measured level increments using 90mm saddle clips.

Figure 4.2: PVC reservoirs for the purpose of ponded head containment

The nature of the experiment insists that loss of ponded water should occur through intended seepage, which results in reduction of ponded water level with time. Further loss through evaporation also contributes to reduction of head. As a result, there is a need for constant replenishment of leachate, however, this is an issue as these rates are not known ahead of time. Consequently the use of a maximum possible total loss rate was estimated to be equal to the maximum possible hydraulic conductivity, which was calculated in a separate experiment detailed in Chapter 3, namely the measurement of flux through application of $CaCl_2$ to compacted soil cores. Thus it was assumed that the total loss rates in the large scale testing would not exceed the measured flux of the sandy loam soil. The need for an outflow (drainage) system is consequent of the inherent unknown actual rate of flow through each of the cores thus making it expensive and difficult to vary the inflow to match this rate in real time. Consequently, a reservoir overflow located at the required height was employed. The expected reduction of HC with time will cause the increase in reservoir depth due to inflow to accelerate, therefore the outflow must be able to account for this i.e. be freely draining. A graphical representation of this concept is detailed in Figure 4.2

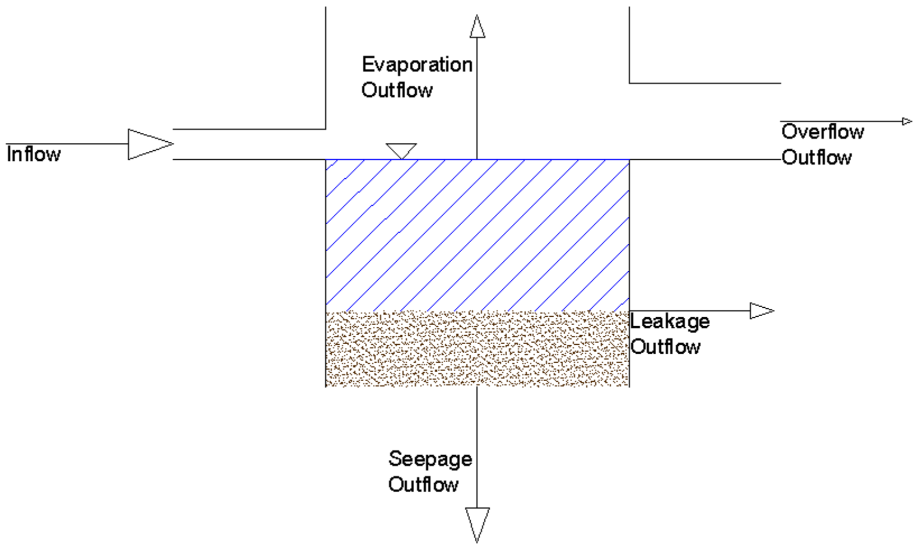

Figure 4.3: Reservoir storage system

The raised nature of the ponded surface level limited the supply methods for the inflow considered in the design process to be either pressure pumps to lift the liquid or an elevated supply reservoir. Due to the frame size constraints previously presented, namely the proximity of the highest ponded head to the roof of the shelter structure, the elevated reservoir was rejected as a design solution. Consequently, an analysis of a pumped supply system was conducted, to determine the pump size required. The analysis used the concept of the Bernoulli equation for pumped pipelines, detailed in Equation 4.2.

$$z_1 + \frac{p_1}{\rho \times g} + \frac{\propto_1 \times V_1^2}{2 \times g} = z_2 + \frac{p_2}{\rho \times g} + \frac{\propto_2 \times V_2^2}{2 \times g} + h_{losses} - h_{pump}$$

Equation 4.2: Bernoulli's hydraulic equation (Nalluri 2009)

The pump was chosen on the basis that it could supply liquid to each of the cores with the parameters of the piped system taken into account i.e. the friction and minor losses. However, due to the drainage height regulation system it was not necessary to specify the exact pump dimensions required, therefore a pump was chosen that would supply more than the expected minimum requirement. This eliminated the need to conduct a difficult analysis using Benoulli's equation.

The pump was required to run continuously for the entire duration of the experiment, therefore a submersible pond pump was decided upon. After research into the brands stocked by local suppliers, it was determined that a Reefe brand submersible pump should be used. Using the rating table provided by Reefe (Appendix B) the required discharge and lift (minimum lift being three metres) were isolated resulting in the RFK5000 pump being considered as suitable. The pumps were installed at ground level, within 44 gallon drum storage reservoirs (Figure 4.4).

Due to low inflow required and connection logistics with the selected pump, a 13 mm irrigation tubing with 3mm open ended distribution drippers was utilised as the pipe flow transport system (Figure 4.3).

Figure 4.4:Inflow distribution system

As depicted in Figure 4.1, there are multiple elevation levels to be serviced by the pumped inflow system. As a result there is a need to be able to regulate the flow rate for each of the elevation levels, this was achieved through the installation of flow reducing valves (Figure 4.4).

Figure 4.5: Independent flow regulation valves

Employing the drainage concept detailed in Figure 4.2, drainage holes were bored at the heights desired. However, for various reasons, the overflow could not simply spill out onto the floor and therefore a drainage system feeding back into the 44 gallon storage drums was installed. Firstly, this required the installation of a nozzle fitting to the bored hole so that the attachment of a 13mm irrigation tube was possible (Figure 4.5). The interface between the fitting and the bored hole required gap filling with waterproof silicon to ensure that all no leaks occurred.

Figure 4.6: Fitted nozzles installed in bored drainage holes

The nozzle-bore interface seal was at risk of breakage due to manipulation of the attached drainage tube, which was required in instances where there was a shallow slope from the nozzle to the central drainage station, depicted on the right in Figure 4.5. In these cases the relatively low flexibility of the irrigation tubing exacerbated this risk, and hence 13mm clear tubing was used instead for the 1:2, and 1:4 columns because of its higher flexibility, again depicted on the right in Figure 4.5.

Subsequently the central drainage was constructed of 90mm standard stormwater PVC, and was fixed in place using 90mm saddle clips in the space between the frame mounting faces, Figure 4.6. In order to reduce cost that would be incurred through longer sections of 13mm drainage, inlet stations where created at the varying elevations. Correspondingly, the larger pipe diameter was selected due to; inlet station size logistics, station openings allowed for nine 13mm drains; economics, 90mm stormwater parts are far cheaper than those of other diameters; combined discharge, the drain had to freely drain the combined total discharge overflow. What's more, the desired station elevations and slopes were achieved through the creation of a stepped system i.e. 90 degree bends achieved through the use of 90 degree joiners fixed in place using PVC primer and cement. Following this the drainage was routed through a hole bored in the side of the 44 gallon storage.

Figure 4.7: Central drainage system, visible open end later routed into storage

4.4 Head and core interface design.

From previous experience had by Bennett and Warren (2015) the use of a smaller stand pipe is fraught with the potential for leaks, thus this new design focused on a better seal. Remaining with the idea of using PVC pipe as the container for the soil we extended this idea to use it as the container for the fluid also. Through cost analysis it was discovered that 90mm storm water PVC was the most economically affordable. Furthermore, due to the length of the proposed pipes, and the required removability it was decided that to soil and fluid should be contained within separate sections of the pipe and then joined together with a 90mm PVC coupling later.

To prevent leaks from escaping through the gap between the coupling and the pipe a number of methods and products were trialled. Joining the coupling to the top head containing pipe using high pressure PVC cement still allowed removability of the core from the apparatus for later testing. However as later discovered cementing whilst the head pipes were attached to the frame caused excess to drip down on the inside of the lower portion of the coupling causing problems when trying to obtain a seal with the soil core.

4.4.1 Vaseline

The first effort to seal the coupling utilised Vaseline, which was thought to provide an adequate seal. This method involved the application of a layer of Vaseline around the male end of the core/coupling interface. After many days work in installing applying the Vaseline and then strenuously lifting and fixing the cores into place with saddle clips and duct tape (a difficult task as the leaching columns are so close together that it is difficult to get a good seal with the duct tape, for future reference it would be better to accommodate more room so that the frame may have been a bit bigger and the columns further apart) it was noticed that much of the Vaseline was scraped out of the gap by the leading edge of the coupling. Upon application of small amount of head to some of the columns it was discovered that the seal was not sufficient. Thus start of leaching was delayed, and another solution to create seal was thought of.

4.4.2 High Pressure PVC cement

Due to the time deadline, a solution that would create a definite seal was thought to be found within the use of PVC cement foregoing the ease of later removal of soil cores. Thus those soil cores used to test seal with the Vaseline were drained and the cores removed cleaned and made ready to be cemented in place. Due to unavailability of assistants when cementing, it was conducted solely by the researcher. Upon the installation and curing of the cement seal it was tested (again by the application of a small amount of head), it was discovered that the seal was not adequate because of the excess drip (mentioned earlier) causing raised ridges. However as a cemented join is of a permanent nature, removal and re cementing was not possible. Thus the head was removed and a solution that could be applied to the external interface between coupling and core was thought of.

4.4.2.1 Water resistant roof/gutter silicon

Following the blunder of the permanent leaky seal from the use of PVC cement, the use of silicon to cover the leading edge of the coupling was trialled. This required the application surface to be adequately cleaned. Firstly by vigorously buffing with Turpentine, and then secondly with the application of PVC cement primer to remove any oils and greases that could potentially loosen the bond between PVC and silicon.

Running a bead of silicon around the front of the leachate column was achieved with ease, yet due to the limited manoeuvrability it was difficult to repeat the process for the rear side of the pipe. Thus the use of silicon applied to a finger was used to apply to the rear. To prevent silicon from adhering to the finger when smoothing the bead into the interface, the appendage was dipped into slightly diluted dish wash liquid. Upon curing of the silicon, and then application of head it was found that nearly of the columns maintained a good seal, however in some the leak was too strong to be held back by externally applied silicon. Thus more silicon was applied whilst the column was still leaking, however this did not work as silicon whilst uncured is too malleable to hold pressurized liquid at bay until cure is achieved. As all columns where filled this time it was not an easy task to drain them and begin again, therefore a solution to stop leaks in progress was sought. Upon research a number of products claiming that they could achieve this feat were discovered.

4.4.2.2 Rustoleum Leak Seal® flexible rubber coating spray

This product is an Aerosol which can simply be sprayed on the area of the leaks origin. It did succeed in mitigating the rate of leakage in some cases, however due to its nature it created a skin over the area under which the leak was contained in a distributed fashion which with time accumulated and eventually lifted the rubber coating off the PVC and the leak continued. Thus another more permanent alternative was needed.

4.4.2.3 Rustoleum Leak Seal® self-fusing tape

This product was quite expensive and fickle, but in cases where the leak was particularly strong the application of this tape created a good seal and the leak was stopped. As mentioned previously with the application of Duct Tape it is difficult to apply tape to the columns and this tape is no exception. The product needs to remain dry to be able to fuse onto itself, and thus application from the top down of the afflicted area is necessary being careful that the hanging tail of the tape does not intercept the path of the leak. In cases where the leak was particularly strong (of which there was only one) another solution had to be utilized.

4.4.2.4 SikaFlex® 227 polyurethane sealant

Upon consultation with a tradesperson the use of this heavy-duty sealant was recommended. It is many times more adhesive than silicon and dries far quicker. Application process is the same as that described in section 4.2.2.1. Unlike silicon the where the malleability of the uncured product allowed the leak to escape through multiple within the medium, this product resulted in only one hole from which the leak could be temporarily re-directed through the use of 3mm irrigation tubing. Upon curing the irrigation tube was cropped and the end plugged with more of the product held in place temporarily with tape.

4.4.3 Future considerations

A lot of time and effort is required to reduce leaks on such an apparatus. Therefore the best solution is to make sure leaks cannot enter the leachate collection device by making sure that the lower end of the soil column (unpressurised) is sealed adequately allowing for excess leaks to bypass and not be counted. Thus for future reference focus should only be on stopping major leaks otherwise too much effort and time will needlessly be wasted, or better still is to cement the coupling to the core first and then the head column thus removing the possibility of gravity causing cement ridges which prevent seal.

5 Chapter 5: Results

5.1 Initial hydraulic conductivity

As presented in prior chapters of this report, there was a need to determine to the maximum hydraulic conductivity (*K*) of the selected compacted soils in order to understand relative reduction in hydraulic conductivity. Calcium Chloride leachate ($CaCl_2$) was matched to the electrical conductivity of the filtered effluent at 5.6 dS/m and the solution used to obtain the maximum conductivity as Ca does not instigate dispersion of soil aggregates. The results of this experiment are presented below (Figure 5.1). Applying a linear regression to the data using Minitab 17 it was found that there was no significant difference in the variance of data throughout time. Hence, steady state conditions were observed and the maximum hydraulic conductivity for the Clay and Sandy Loam was $1.92207\text{x}10^{-8}$ and $6.00563\text{x}10^{-8}$ m/s. These initial conductivities were achieved by compacting the respective soil samples to 98% of the maximum dry density, accounting for such low initial hydraulic conductivities.

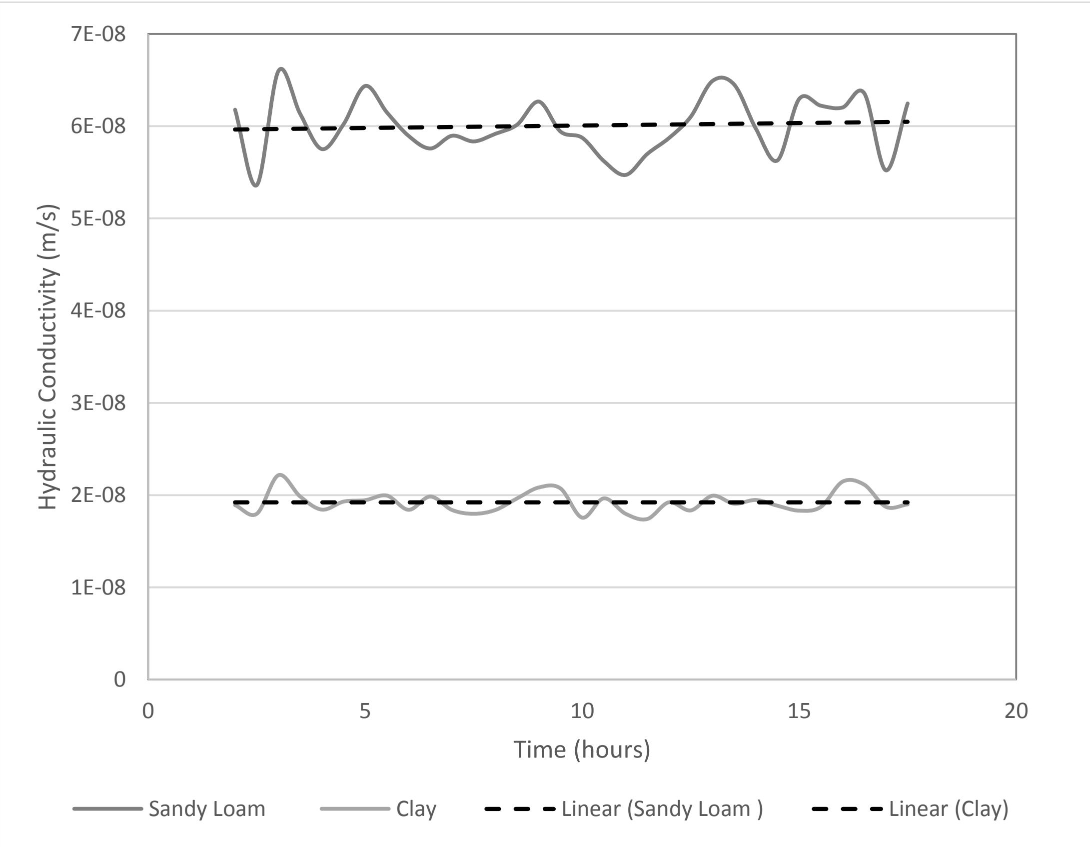

Figure 5.1: Comparison curve of flux of soils subjected to calcium chloride leachate

From Figure 5.1, it can be clearly seen that the Sandy-Loam soil had initial hydraulic conductivity approximately three times greater than that of the clay. This was an expected result given the highly established linkage between soil texture and hydraulic conductivity.

5.2 Total suspended solids

Total suspended solids was measured to understand the amount of colloidal material within effluent that remained within the soil cores after leaching. Initially the effluent contained 2.06 g/L of suspended solids and these solids were all considered to be suspended in the true definition of a colloidal suspension. This was ensured by >200 h of settling, which ensured the remaining total suspended particulate was <2 μm from the depth effluent was capture, according to Stokes Law. A colour comparison between the leachate samples and the filtered effluent gave indication that entrainment of suspended particulate was occurring in the soil liner, and that this was occurring from as early as 24 hours post permeate application (Figure 5.3).

Figure 5.2: Colour comparison of leachate (right) with effluent (left). Time intervals of sample collection are 0, 259, 477 and 815 hours (left to right) after application of head to cores

Total suspended solids (Figure 5.3) contained within the leachate over time was compared to that of the filtered effluent throughout the experiment. Note that the measurement at zero time is that of the filtered effluent prior to entering the leaching column. A power curve function was fitted to the trend suggesting that as time increases towards 1739.5 h (time of leaching experiment) that discharged total suspended solids approaches 0.46 g/L.

This trend line explains 79% of variation in the data with time, fitting the observed data to a very high level (R=0.89; R^2=0.79).

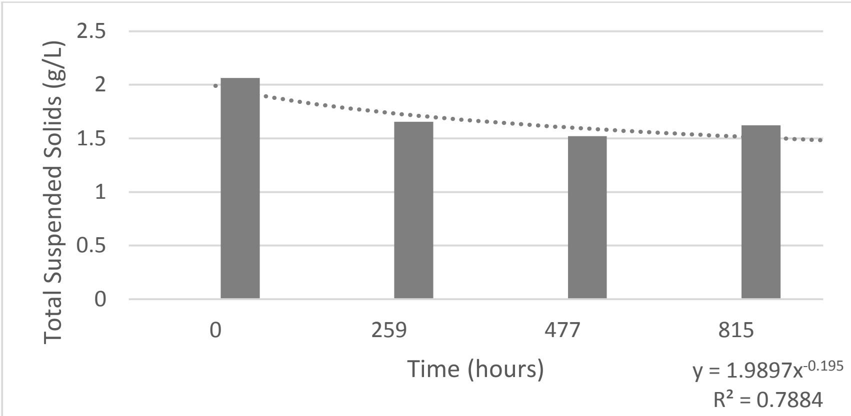

Figure 5.3: Total Suspended Solids TSS (g/L) measured at different time intervals. Time point zero, is that of the effluent, the other time points are that of the leaching fluid.

5.3 Hydraulic conductivity following application of filtered effluent

Hydraulic conductivity was measured over time to investigate how colloidal particulate reduces the hydraulic conductivity and what timeframe this might be expected to occur in. Furthermore, it allowed comparisons between scale effluent pond characteristics with the same hydraulic gradient. Figure 5.2 shows these changes in hydraulic conductivity with time and application of effluent. From statistical error analysis of different time points in the time series, it was found that hydraulic conductivity between the scales was not significantly different for the Sandy-Loam (using a 95% confidence interval). The clay sample effluent pond simulations at a scale of 1:1 and 1:2 did not leach during the experimental time period. Hence comparison of the 1:4 scale to these is not possible. It is, however, possible to state that the hydraulic conductivity for the sandy-loam columns are significantly different from the hydraulic conductivity of the 1:4 clay column replicates.

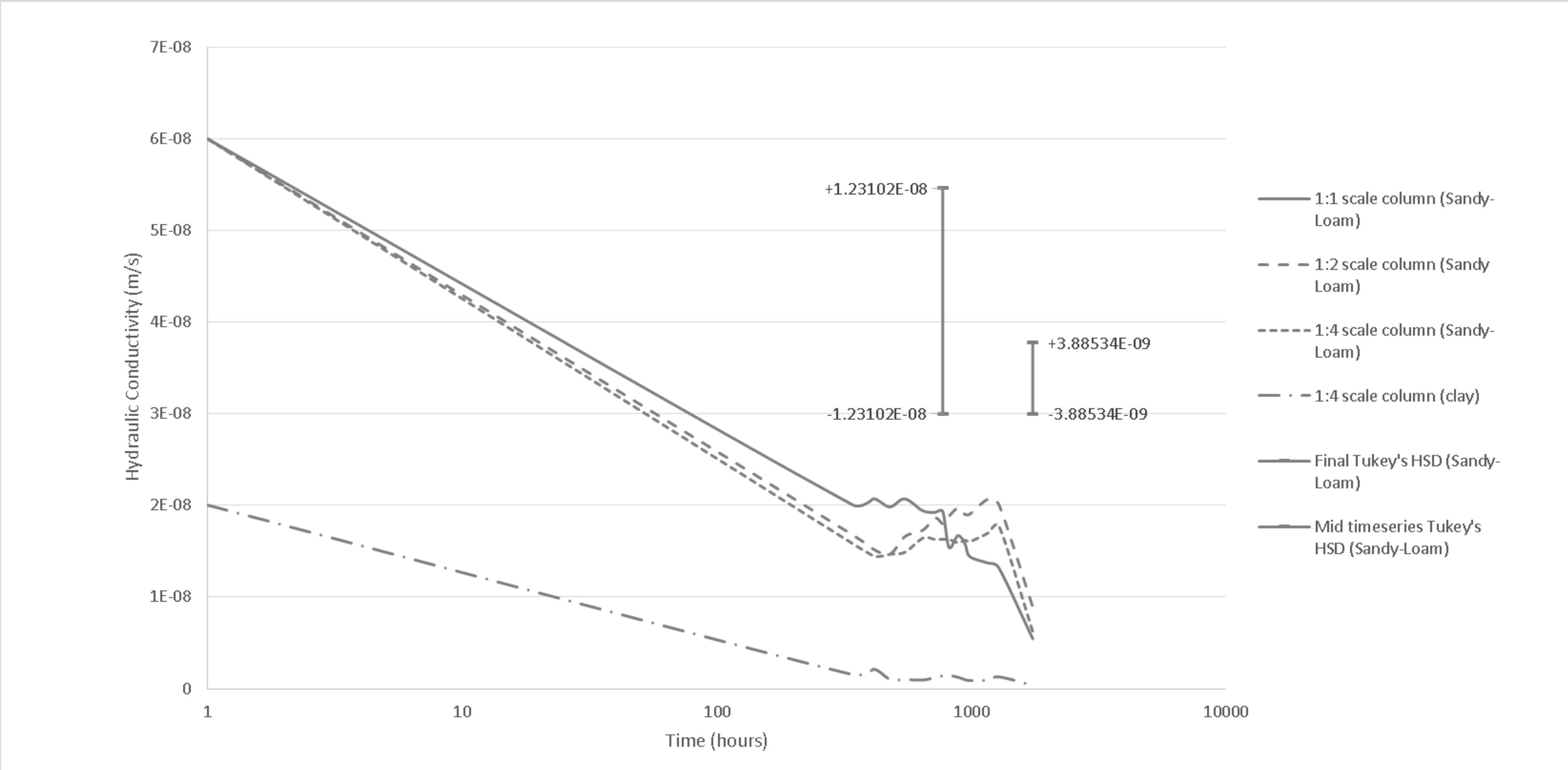

Figure 5.4: Comparison of hydraulic conductivity between effluent leachate column scales

5.4 Soil liner saturation depth

It is known that the effluent contains suspended solids, and established that these affect hydraulic conductivity through reduction in porosity dimensions. Hence, soil liners were investigated to identify the depth at which saturation conditions ceased and flow through the remaining liner depth was at unsaturated conditions. Following the final HC measurement at 1739.5 hours a core from each of the three scales of Sandy-Loam soil was selected and holes bored incrementally from the bottom up; bored holes represented a large macropore and would only conduct effluent if the liner was under saturated conditions Hence, the determination of the depth of saturation was binary and discovered by examination of the relative location of the bored hole from which liquid leached to the top of the soil core (Table 5.1). Statistical analysis was not possible as only one replicate was investigated to maintain the integrity of the experiment as it is planned to continue until one year has passed. Saturated conditions were observed between 33 and 47% depth of the soil liner, which equated to conditions occurring at 210, 75 and 45 mm for the 1:1, 1:2 and 1:4 scales, respectively.

Table 5.1: Depth of Saturation comparison between column scales

Sandy-Loam Column Scale	***Depth of Saturation (mm)***	***Relative Depth of Saturation (%)***
1:1	210	46.7
1:2	75	33.3
1:4	45	40.0

6 Chapter 6: Discussion

6.1 Importance of liner depth in colloidal entrapment

The study by Bennett and Warren (2015) established that pore blockage due to livestock effluent colloid entrainment does occur. However, their findings are confined to the analysis of the effect on one liner thickness size, which was also only a shallow liner of 50 mm. The goal of this study was to investigate the role of liner thickness in hydraulic conductivity reduction for the purpose of determining whether cost can be mitigated in the installation of compacted *in-situ* soil liners for intensive livestock effluent ponds. It was hypothesised the effect of liner depth on colloid induced pore blockage to be important, due to the increased tortuosity and probability of colloidal adhesion within the soil network.

The results support the hypothesis that liner thickness is indeed significant, which is in direct contrast of findings for non-colloidal effluents (Cihan et al. 2006; Tyner and Lee 2004). However, this is not evident from the plot depicted in figure 5.4, which shows that hydraulic conductivity is independent of liner thickness in the compacted coarse-textured soil. When considering the depth of saturated zone within the soil liner (Table 6.1) it can be deduced that the depth of saturated zone is a function of hydraulic pressure head, meaning that the saturated zone is variable with pressure. Hence, it is also deduced that depth of liner is important as saturated depths represent zones of colloidal entrainment whereby unsaturated depths below the entrainment zone are of higher conductivity due to pores not being blocked by colloidal particulate (Bennett and Warren 2015; Hillel 2004). Figure 6.1 conceptualises the mechanisms and illustrates the effluent pond system as it is observed to be operating.

Based on these observed differences it is clear that the effluent pond simulated systems are operating as two layer systems in the liner depth, where the unsaturated depth is no longer controlling hydraulic conductivity (Figure 6.1). Hence, the hydraulic gradients ($\Delta H/L$) are recalculated as 12.90, 17.67 and 14.89, for scales 1:1, 1:2 and 1:4, respectively. Based on this, it is thus prudent to recalculate the hydraulic conductivity information presented in Figure 5.4 on this basis. We expect that the colloidal blockage occurs in a gradient within the saturated zone, with the highest concentration near the surface of the liner gradually decreasing with depth. However, there is no way of determining this gradient from the existing data. Therefore the application of the filtercake theory adjustment to the HC curves is based on the assumption that the gradient does not exist, and that the depth of the filtercake is constant throughout the saturated depth. Figure 6.2 depicts the simplified saturated depth system adjusted for the differing hydraulic gradients.

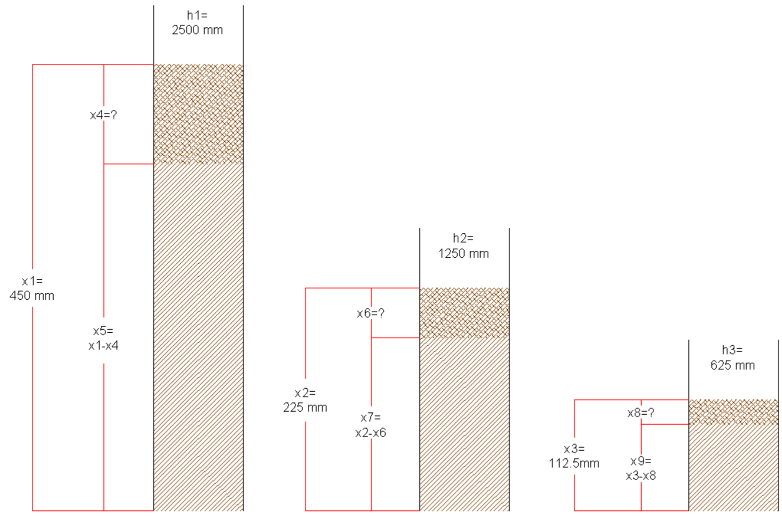

Figure 6.1: Diagram of depth of pore blockage layer expected between scales

Adjusting for the change in hydraulic gradient (Figure 6.1) demonstrates that hydraulic conductivity decreases with reduction in liner thickness as:

$$K = \frac{VL}{At\Delta H} = q\frac{L}{H}$$

Where *q* can be treated as a constant because the change in hydraulic head and liner length does not affect volume discharge (*V*), the cross-sectional area of flow (*A*) or the time this was observed for (*t*). Taking into account ~40% functional liner thickness in terms of saturation and the effect this has on the saturated hydraulic head (approximate 1.2% reduction), we see that:

$$K_{adj} < K,\ \text{where:}$$

$$K_{adj} = q\frac{0.4L}{0.998H}$$

Taking this into account it is found that adjusted hydraulic conductivity at 1739.5 hours was X, Y and Z, for the 1:1, 1:2 and 1:4 scale effluent simulations, respectively (Figure 6.2). Furthermore, this represents a reduction in calculated hydraulic conductivity at 1739.5 h of X1, Y1 and Z1%, respectively. Hence, it is demonstrated that liner thickness is important in limiting discharge with increase in hydraulic head (Figure 6.1) and that this depth is important in making assessment of how effective liners are. The K_{adj} function would be useful in determining true conductivity for the clay loam, but would require further calibration for other soils, thereby the coefficients of 0.4 and 0.998 should be treated as generic and optimisable:

$$K_{adj} = q\,\frac{aL}{bH}, \text{ where } a \gg b$$

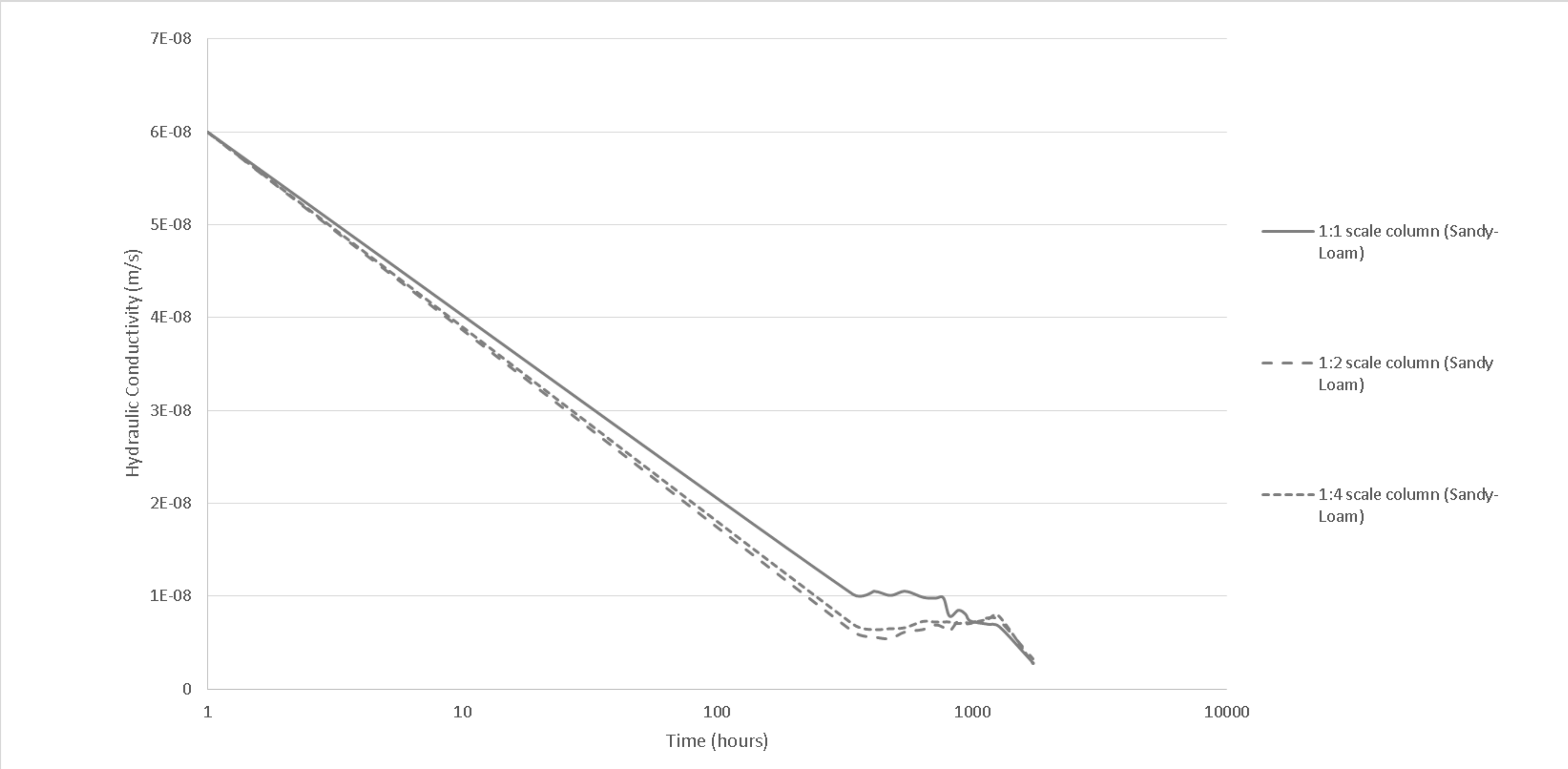

Figure 6.2 Speculative hydraulic conductivity with hydraulic ratio adjusted according to filtercake theory

6.2 Efficiency of colloidal entrapment

From the comparison of the TSS content within the effluent and the leachate, it is evident that there is a reduction between the two. However further significant change through reduction with time does not occur, although fits highly a decreasing logarithmic trend towards 0.46 g/L discharged with leachate at 1739.5 h. This suggests that the difference in TSS content is being retained within the soil, although with the effects of dispersion it is noted that the results might be affected by the presence of particles of the subject soil which have been dispersed and also been leached through. Turbidity measurements suggested this soil was dispersive spontaneously and mechanically. That said, this would be expected to be a minor effect compared to the retention of particulate (McGechan 2002; McGechan and Lewis 2002). Results from the present study support the findings of Bennett and Warren (2015), who also noted that there was no significant acceleration or deceleration of the removal of TSS from the effluent with time.

The depth of the saturated boundary layer suggests that it alone is the depth to which the colloids become entrained. However, we might reasonably expect there to be colloids entrapped throughout the entire liner thickness, as the saturated zone merely indicates that it has a higher concentration of entrained colloids leading to governance of the permeability for the entire liner. Given this speculation, we can expect that colloids within the saturated zone are entrained in gradient, where the highest concentration occurs near the surface and the lower boundary being indefinite, or diffuse. We expect the lower boundary to be diffuse, due to the non-homogenous nature of soil pore networks. However, it is suggested that this gradient must be relatively small in order that upper layers not restrict saturation conditions.

This requires further research, namely the dissection into layers of the leached cores, so that with the application of suction it is expected that the extent of pore blockage may be determined; i.e. determination of pore size distribution via weight of retained water with applied increasing suction.

6.3 Management implications

These experimental results clearly show that liner depth is important to be considered where effluents are dominated by colloidal particulates, and that the crucial depth of liner is related to the size of the hydraulic head. From a management perspective this insinuates that these results are only applicable where filtration systems and/or sediment ponds are used prior to effluent storage in dedicated effluent ponds. Where effluents contain significant fractionation of particulates that are non-colloidal, it is apparent that liner depth is not important (Cihan, Tyner & Wright 2006; Lado, Ben-Hur & Assouline 2005; Meyer, Olson & Baier 1972; Tyner & Lee 2004). However, Bennett and Warren (2015) have shown that filtercake theory still explains hydraulic reduction in both instances as irrespective of whether particulate pore blockage occurs within the soil liner itself, or via a sludge layer (filtercake) atop the liner, it is still the particulates that are physically responsible for the blockage. Hence, it is deduced that filtercake theory will hold for these experimental data also.

From the results it is deduced that liner thickness as a function of applied pressure head, is indeed important to the colloidal mechanism for pore blockage. Thus this means that feedlot operators must continue to heed the guidelines of pond construction, and install liners of the recommended thickness in order to reduce their beneath pond seepage rates to the guideline rate of $1.0x10^{-9}$ m/s or 31.5 mm/year. It was found that the 1:4 scale clay soil liner reached this rate at approximately 900 hours, on the contrary the sandy-loam soil in all scales did not achieve the guideline rate, but with projection of the trend it is expected to in the near future, confirming the findings of Bennett and Warren (2015) that *in situ* soil materials should be appropriate for pond bases within the Sandy Loam through Clay textures.

Liner depth contributes further importance in seepage management, in that during the initial phase of effluent application the hydraulic conductivity is governed by the soil unaltered by the colloid deposition, thus there is increased seepage due to higher HC. If the liner was of no importance in regards to HC after establishment of colloidal blockage, we would still expect regulatory bodies to impose the same liner depth regulation to avoid contamination of the groundwater in the early stages of the pond use.

For the Sandy Loam it was demonstrated that coefficients of hydraulic gradient reduction would be useful in calculating the expected true reduction in hydraulic conductivity. These coefficients are not applicable to other soils without further investigation, hence a generic form of the equation was presented. In order to inform the extent of reduction expected for effluent pond *in situ* base material it would be pertinent to explore the range of these coefficients with texture and timexTSS (i.e. increasing opportunity for particulate blockage).

Two soils were initially investigated, yet only the sandy-loam soil provided flux for all scales of the effluent pond systems. From the initial hydraulic conductivity experiment it can be seen, as was expected, that the clay soil had a much lower initial HC. However, the result is more likely attributed to effluent being applied to soils in the unsaturated state, which is representative of the initial condition of pond liners in reality. Irrespective of the fact the Clay soil has not yet saturated, this informs an important discussion: Current calculations on liner thickness are based on the initial inherent conductivity, which ignores the effluent particulates and assumes the depth is required for this initial hydraulic conductivity to limit discharge (flux). Literature for colloidal and non-colloidal effluents has shown that only considering the medium's internal initial properties is not adequate to understand the extent of hydraulic reduction (e.g. Cihan et al. 2006; Bennett and Warren 2015).

However, one might consider hydraulic conductivity to be a misleading measure, and that flux is a more representative parameter to use as the regulation threshold. The contrast between the soil textures, and the fact the data from the actual 1:1 scale suggests that flux is the real threshold we should consider, as this allows incorporation of the colloidal properties and the pressure head. Contrary to the hydraulic conductivity, which is a proportionality factor describing an inherent property of the medium, and does not allow adequate consideration of colloidal properties and pressure head. Furthermore, if the real concern is discharge of nutrients and salts to the environment beneath the ponds, then flux is the more adequate threshold parameter. This is due to the difference in magnitude of seepage into the groundwater between ponds of equal hydraulic conductivity, or flux, yet different pond impact area.

7 Chapter 7: Conclusion

7.1 Conclusions of results

The purpose of this research project was to further the research done by Bennett and Warren (2015) in investigating the ability for colloids within effluent to cause pore blockage, and as a result a reduction in seepage. However, this project differed from it, in that the main aim was to investigate the role of liner thickness on pore blockage.

It was discovered that effluent pond liner thickness is of significance in reducing hydraulic conductivity. This conclusion was drawn through consolidation of results regarding the difference in hydraulic conductivity between three differing scales of hydraulic gradient, and also the assessment of the depth of saturation caused by the imposition of a layer of lower porosity through the deposition of colloids in the soil pore network. This means that feedlot operators will have to continue to follow the industry best practice procedures of establishing liner thickness, although future work should be able to parameterise the ratio of liner depth to hydraulic head meaning liner depth could be customised based on soil texture and pressure head.

In addition to these findings, the results of Bennett and Warren (2015) were confirmed with regard to hydraulic conductivity in Sandy Clay to Clay textured soils being suitably reduced with time and effluent TSS to achieve conductivity , below the guideline rate of 1.0×10^{-9} m/s proposed by Skerman (2000).

7.2 Future work

As discovered, this report has provided some insight into the influence of liner thickness on colloidal blockage induced seepage reduction. It is recommended that there be further investigation into the colloidal entrainment gradient of the soil to, to determine the distribution of particles throughout the core and to allow predictive capability of pore size distribution decrease with increasing cumulative length of infiltrating TSS.

It is further recommended that the hydraulic conductivity results be compared with that of a synthetic replicate of the effluent, to isolate the contribution to HC reduction of the colloidal blockage as demonstrated in (Bennett & Warren 2015).

Linking this work with previous work should allow numerical modelling procedures to be used to predict hydraulic reduction on the basis of soil texture, time and TSS. The model

HYDRUS uses such an approach for solutes, heat and water transport, but does not currently have a reactive colloidal

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Appendix A – Project specification

University of Southern Queensland

FACULTY OF HEALTH, ENGINEERING AND SCIENCES

ENG4111 and ENG4112 Research Project

Project Specification

FOR: **Timothy Colin GEORGE**

TOPIC: Effect of soil depth on Darcy's coefficient of hydraulic conductivity as influenced by effluent suspended particulate

SUPERVISOR: Dr John Bennett, NCEA

SPONSERSHIP: No Sponsorship

PROJECT AIM: To determine if soil depth increases likelihood of effluent suspended particulate adhesion, thus decreasing saturated soil hydraulic conductivity. Prior research based on filtercake theory suggests that clay liner depth is not important as the filtercake controls hydraulic conductivity, but recent USQ research challenges this showing suspended particulate to be of diameter less than soil pore size. Hence, the aim tests this.

PROGRAMME:

1. Literature review
2. Gather uncontaminated soil, as well as some livestock effluent.
3. Compact soil into cylinders at varying soil depths. Fill the cylinders with effluent maintaining a constant hydraulic gradient for all effluent head heights and soil depths. Pump water through the system, and collect and weigh the exuded water. This continues until water flow reduces to below existing governing body standards.
4. Using Darcy’s equation for hydraulic conductivity determine the hydraulic conductivity.
5. Determine zones of unsaturated flow within soil depth at conclusion of experiment.
6. Compare results with those generated in previous USQ and external research projects
7. Validate result.

8. Submit an academic research dissertation

T'C'George (Student) Dated: 18/03/2015

____________________ (Supervisor) Dated: / /2015

Examiner/Co-Examiner ________________________

AGREED:

Appendix B – Data sheets

In the design methodology there was a requirement to undertake a pump selection for a submersible pump from Reefe Pumps, Australia (Figure 7.1).

Head (m)	Approximate Flow Rate in Litres per Hour (without restriction)						
5.5							0
5.0							950
4.50					(0 @ 4.5m)		1750
4.25					350		2250
4.00					750	(0 @4.0m)	2750
3.75					1050	450	3200
3.50					1350	900	3650
3.25					1600	1350	4000
3.00					2000	1800	4350
2.75				(0 @ 2.75m)	2250	2000	4655
2.50			(0 @ 2.4m)	400	2600	2450	4960
2.25			300	1000	2800	2900	5410
2.00	(0 @ 2.0m)	(0 @ 2.0m)	600	1300	3100	3350	5860
1.75	200	250	850	1650	3300	3500	6210
1.50	400	500	1150	2000	3500	3950	6560
1.25	650	700	1350	2400	3700	4350	6905
1.00	700	850	1600	2700	4000	4800	7250
0.75	825	1000	1750	3000	4200	5035	7515
0.50	950	1200	2000	3400	4400	5270	7780
0.25	1075	1350	2200	3600	4600	5550	8140
0.00	1100	1500	2300	4000	5100	6000	8500
MODEL	RFK1100	RFK1500	RFK2400	RFK4000	RFK5000	RFK6000	RFK8500

Figure 7.1: Pump selection table provided by Reefe Pumps

Appendix C – Raw data and specific software output

Minitab 17 was used to analyse the error within the collected data, and to analyse variance. This appendix provides sample outputs from the program. Firstly we see the output of "honest significant difference" of the hydraulic conductivity at the mid-point between the three different scales that had been subjected to leaching with filtered effluent.

One-way ANOVA: 770hrs HC versus Treatment

```
Method

Null hypothesis         All means are equal
Alternative hypothesis  At least one mean is different
Significance level      α = 0.05
Rows unused             6

Equal variances were assumed for the analysis.


Factor Information

Factor     Levels  Values
Treatment       4  R1, R2, R3, R4
```

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Treatment	3	882.5	294.17	6.01	0.008
Error	14	685.8	48.98		
Total	17	1568.3			

Model Summary

S	R-sq	R-sq(adj)	R-sq(pred)
6.99882	56.27%	46.90%	33.10%

Means

Treatment	N	Mean	StDev	95% CI
R1	6	19.32	10.03	(13.20, 25.45)
R2	4	17.96	3.43	(10.45, 25.47)
R3	4	16.35	6.99	(8.84, 23.85)
R4	4	1.446	0.329	(-6.060, 8.951)

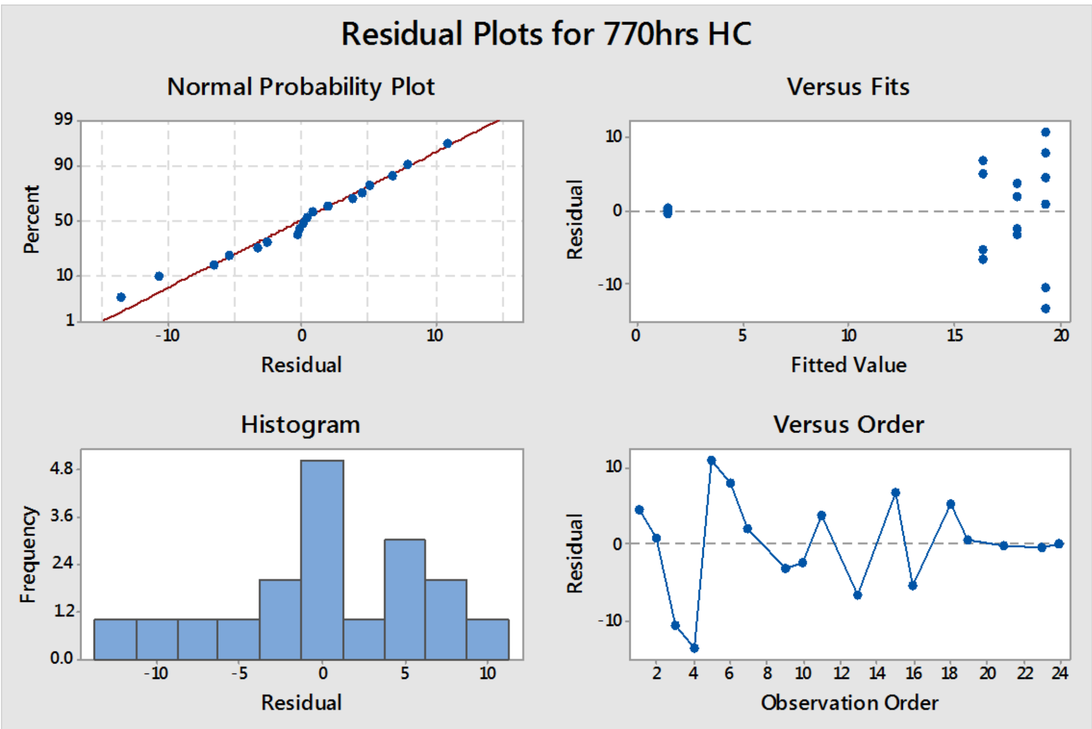
Pooled StDev = 6.99882

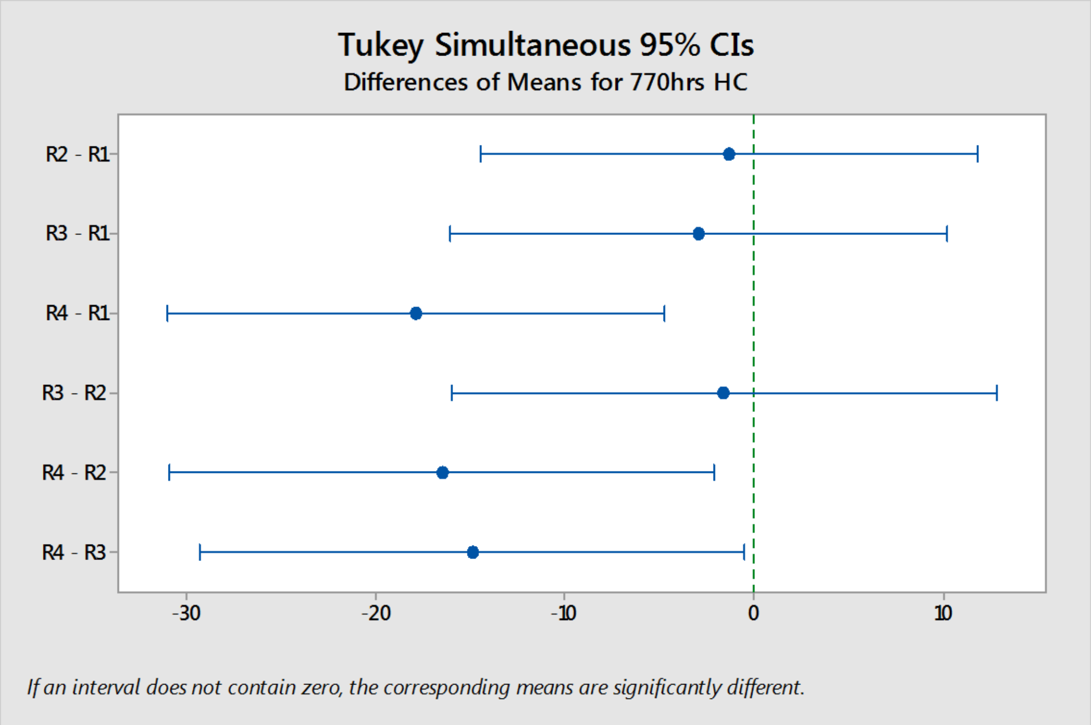
Tukey Pairwise Comparisons

Grouping Information Using the Tukey Method and 95% Confidence

Treatment	N	Mean	Grouping
R1	6	19.32	A
R2	4	17.96	A
R3	4	16.35	A
R4	4	1.446	B

Means that do not share a letter are significantly different.





Therefore the sandy-loam column results, at the 770hour mark, are not significantly different. However the 1:4 clay column result is significantly different from the sandy-loam results.

One-way ANOVA: 1739.5hrs HC versus Treatment

Method

Null hypothesis	All means are equal
Alternative hypothesis	At least one mean is different
Significance level	$\alpha = 0.05$
Rows unused	4

Equal variances were assumed for the analysis.

Factor Information

Factor	Levels	Values
Treatment	4	R1, R2, R3, R4

Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Treatment	3	193.00	64.332	15.05	0.000
Error	16	68.40	4.275		
Total	19	261.40			

Model Summary

S	R-sq	R-sq(adj)	R-sq(pred)
2.06768	73.83%	68.93%	58.39%

Means

Treatment	N	Mean	StDev	95% CI
R1	6	5.46	2.75	(3.67, 7.25)
R2	4	8.75	2.19	(6.56, 10.95)
R3	4	6.14	2.30	(3.95, 8.33)
R4	6	0.3118	0.1602	(-1.4777, 2.1013)

Pooled StDev = 2.06768

Tukey Pairwise Comparisons

Grouping Information Using the Tukey Method and 95% Confidence

Treatment	N	Mean	Grouping	
R2	4	8.75	A	
R3	4	6.14	A	
R1	6	5.46	A	
R4	6	0.3118		B

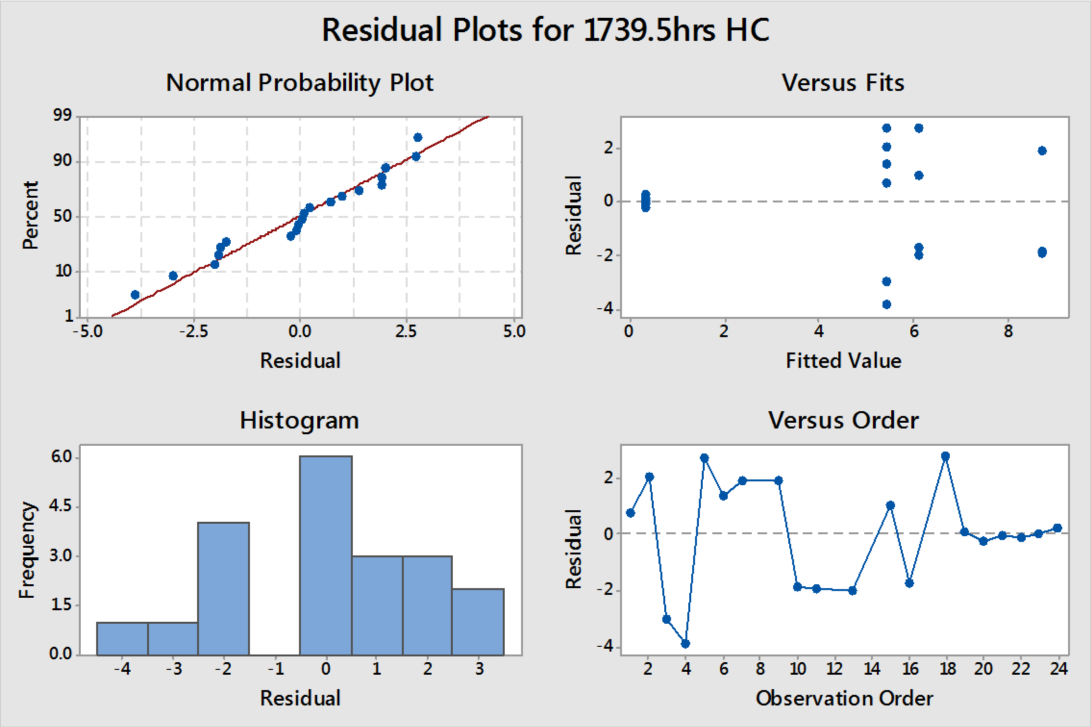
Means that do not share a letter are significantly different.

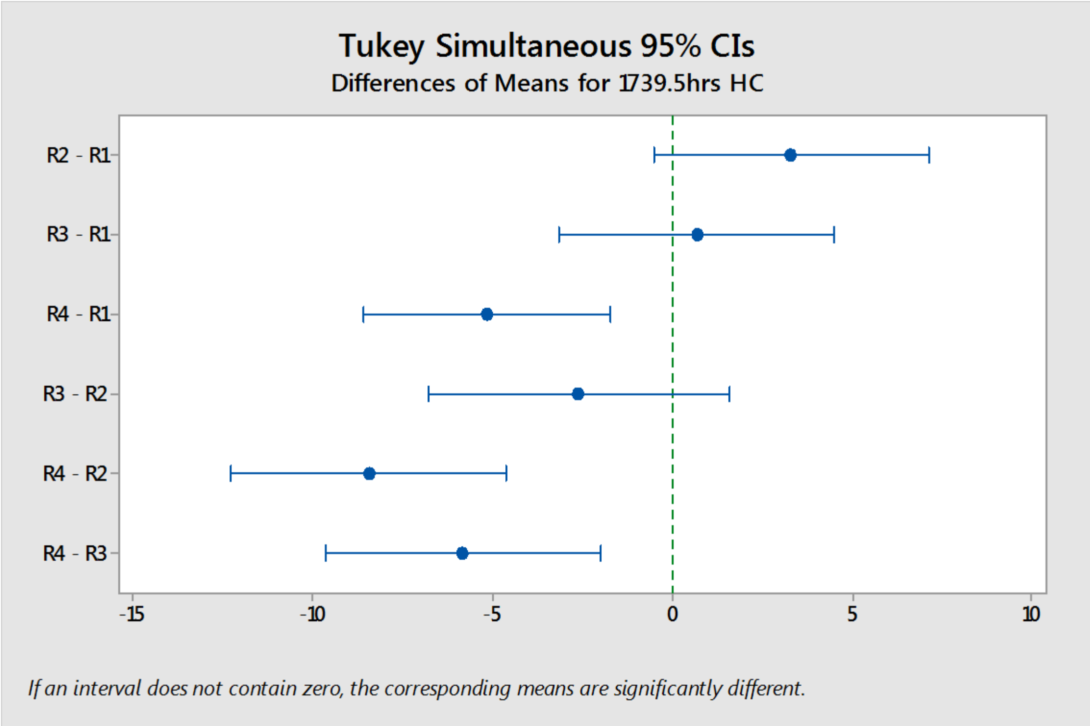
Tukey Simultaneous Tests for Differences of Means

Difference of Levels	Difference of Means	SE of Difference	95% CI	T-Value	Adjusted P-Value

```
R2 - R1          3.30      1.33  ( -0.53,  7.12)    2.47    0.103
R3 - R1          0.68      1.33  ( -3.14,  4.50)    0.51    0.955
R4 - R1         -5.15      1.19  ( -8.56, -1.73)   -4.31    0.003
R3 - R2         -2.62      1.46  ( -6.80,  1.57)   -1.79    0.314
R4 - R2         -8.44      1.33  (-12.26, -4.62)   -6.33    0.000
R4 - R3         -5.83      1.33  ( -9.65, -2.00)   -4.37    0.002

Individual confidence level = 98.87%
```





Sample of raw data from effluent leaching columns

	Date	04-09-15	07-09-15	11-09-15	14-09-15	16-09-15	18-09-15	21-09-15	23-09-15	25-09-15	02-10-15	07-10-15	15-10-15
	Time	9:35:00 AM	6:55:00 AM	6:45:00 AM	9:45:00 AM	3:15:00 PM	11:28:00 AM	7:45:00 AM	6:00:00 PM	12:00:00 PM	3:45:00 PM	4:30:00 PM	12:00:00 AM
	hours	65.75	69.333333	95.8333333	75	53.5	44.2166667	68.2833333	58.25	42	171.75	120.75	463.5
	Seconds	236700	249600	345000	270000	192600	159180	245820	209700	151200	618300	434700	1668600
	hours elapsed	477.08333	546.41667	642.25	717.25	770.75	814.966667	883.25	941.5	983.5	1155.25	1276	1739.5
Ver	**mass**	0	0	0	0	0	0	0	0	0	0	0	0
2500:450	**Comments**												
1	**q**	0	0	0	0	0	0	0					
	K	0	0	0	0	0	0	0					
Ver	**mass**	0	0	0	0	0	0	0	0	0	0	0	0
2500:450	**Comments**												
2	**q**	0	0	0	0	0	0	0					
	K	0	0	0	0	0	0	0					
SL	**mass**	221.85	224.63	290.6	224.94	180.9	106.75	172.37	138.31	93.69	376.29	260	406.66
2500:450	**Comments**												
3	**q**	1.559E-07	1.497E-07	1.4008E-07	1.3855E-07	1.562E-07	1.1153E-07	1.1661E-07	1.0969E-07	1.0305E-07	1.012E-07	9.947E-08	4.053E-08
	K	2.378E-08	2.283E-08	2.1368E-08	2.1134E-08	2.3827E-08	1.7012E-08	1.7788E-08	1.6732E-08	1.5719E-08	1.544E-08	1.517E-08	6.1825E-09
SL	**mass**	293.14	288.66	337.69	246.4	152.87	106.47	207.43	166.81	108.65	390.71	260.29	492.43
2500:450	**Comments**												
4	**q**	2.06E-07	1.923E-07	1.6278E-07	1.5176E-07	1.32E-07	1.1123E-07	1.4033E-07	1.3229E-07	1.195E-07	1.051E-07	9.958E-08	4.9078E-08
	K	3.142E-08	2.934E-08	2.483E-08	2.3151E-08	2.0135E-08	1.6968E-08	2.1406E-08	2.0179E-08	1.8229E-08	1.603E-08	1.519E-08	7.4865E-09
SL	**mass**	64.67	77.49	100.86	80.7	65.61	31.95	56.09	49.43	33.55	159.34	113.56	163.19
2500:450	**Comments**												
5	**q**	4.544E-08	5.163E-08	4.8618E-08	4.9705E-08	5.6651E-08	3.3379E-08	3.7946E-08	3.92E-08	3.6901E-08	4.286E-08	4.344E-08	1.6264E-08
	K	6.931E-09	7.876E-09	7.4163E-09	7.5822E-09	8.6417E-09	5.0917E-09	5.7883E-09	5.9797E-09	5.6289E-09	6.537E-09	6.627E-09	2.481E-09
Ver	**mass**	0	0	0	0	0	0	0	0	0	0	0	0
1250:225	**Comments**												
6	**q**	0	0	0	0	0	0	0					
	K	0	0	0	0	0	0	0					
Ver	**mass**	0	0	0	0	0	0	0	0	0	0	0	0
1250:225	**Comments**												
7	**q**	0	0	0	0	0	0	0					
	K	0	0	0	0	0	0	0					