Experience as a Guide to Geotechnical Practice In Winnipeg

by

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ABSTRACT

The products of our engineering works are everywhere and the safety and satisfactory performance of these works is the result of the deliberate inclusion of experience based judgement. The starting point for many of these works is the recognition of what has been done in the past, both successfully and unsuccessfully. Often however, a geotechnical engineer must rely on theory and to some degree, experimentation, in particular when advancing new methods of design and construction. This can be daunting when one considers that the natural materials we work with are highly complex with large variability and in some cases, uncertainty in their properties and behaviour. Analysis and decision making based on these properties includes a mix of theoretical and empirical techniques, requiring significant engineering judgement.

The experience gained and lessons learned by geotechnical practitioners in the early 1900s are as valuable today as they were at that time and this becomes the hypothesis upon which the Author has prepared this document. It is often that a problem encountered today, was encountered and very likely solved in the past and that its resolution at that time can be applied (at least in principle) to modern practice. Little in engineering happens by chance and many failures are the result of a poor decision based on poor information, an underestimation of natural forces and laws of physics, or perhaps, the overestimation of one's experience and ability. Successes on the other hand are often a culmination of supplementing intuition with experienced based judgement. Once one understands and appreciates the importance of taking advantage of the past experience of others and using this experience as a guide forward, then improved judgement and capacity for professional practice will result.

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1 INTRODUCTION

Many years ago, Karl Terzaghi advised his class at Harvard University

> *"Engineering is a noble sport... but occasional blundering is a part of the game. Let it be your ambition to be the first one to discover and announce your blunders... Once you begin to feel tempted to deny your blunders in the face of reasonable evidence you have ceased to be a good sport. You are already a crank or a grouch."* (Goodman 1999).

This was excellent advice in an era when the science of geotechnical engineering was not well understood and the breadth of collective experience was limited. While Terzaghi's acceptance of mistakes in pursuit of advancement is the cornerstone of innovation and should never be discouraged, today's junior engineers would also be well advised to take advantage of the years of experience from local practitioners - look upon this experience as a helpful guide to practice and in doing so, perhaps avoid repeating a blunder of the past.

In preparation for undertaking this thesis, the Author has researched information related to the field of geotechnical engineering focussing on the City of Winnipeg from as early as the turn of the 20th century. Questions that arose from this research were how do we best transfer knowledge and experience from one generation of engineers to the next? How do we, as designers, really know how large a safety factor we have unless the boundaries of failure are tested? Are design errors and failures unavoidable? How can we use past experience and case histories to improve reliability? The answers are clear - once one understands and appreciates the importance of taking advantage of the past experience of others and using this experience as a guide forward, then improved judgement and capacity for professional practice will result.

This thesis draws on the Author's 30 years of experience in geotechnical engineering, during which time he has formed an understanding of what works well in local practice and where difficulties may perhaps be encountered. The intent is to provide his experience as a bridge between well-established theory and local practice and help young geotechnical engineers better predict performance based on the use of realistic design parameters, theoretical models and construction considerations. It is offered with the hope that engineers early in their chosen profession will embrace creative thinking but understand the limitations of their experience relative to others.

Geotechnical engineers are faced with the daunting task of working with highly complex natural materials with large variability and in some cases, uncertainty in their properties and behaviour. Analysis and decision making based on these properties includes a mix of theoretical and empirical techniques, requiring significant engineering judgement. Beyond being able to measure the properties and perhaps behaviour of small samples of soil or rock, a typical sub-surface investigation provides but a glimpse of the conditions that lie beneath the ground surface. *"Site Characterization: Expect the unexpected"* was the theme of an unpublished presentation by Professor Ralph Peck to the Winnipeg chapter of the Canadian Geotechnical Society in 2000. This valuable lesson is a reminder that even with the best of investigative techniques, unexpected conditions should be expected. The following quotation from Professor Ralph Peck so eloquently describes this reality:

> *"Nature did not follow standards in creating the mass of rock or soil in question. A defect or a field condition potentially fatal to the performance of a project may exist that escapes the standard investigation. Experience leading to judgement is the best defence against the consequences of such a possibility, and the course of action*

leading to an appropriate solution will differ amongst individuals of different experience. That is, judgement is an essential ingredient in geo-engineering and it cannot be standardized".

The experience gained and lessons learned by geotechnical practitioners in the early 1900s are as valuable today as they were at that time. The same holds true for the present day practitioner whose experience and judgement are valuable assets in engineering - this becomes the hypothesis upon which the Author has prepared this document. Although the passage of time has relegated some of the early case histories into the archives, their value has not diminished. Ironically, it is sometimes only through years of experience that we begin to appreciate the relevance of such historical information. It is often that a problem encountered today, was encountered and very likely solved in the past and that its resolution at that time can be applied (at least in principle) to modern practice - lessons learned from failures can be more valuable than those learned from successes.

The guidance provided herein is not intended to replace or discourage the use of intuition which includes the very important personal sense of what will work and what will not work - this is human nature. But worth remembering is that little in engineering happens by chance and many failures are the result of a decision based on poor information, an underestimation of natural forces and laws of physics, the overestimation of one's experience and ability, or failure to recognize a problem when it occurs. Successes on the other hand are often a culmination of supplementing intuition with experienced based judgement. Don't be afraid to seek more senior advice, re-evaluate design parameters and apply the observational method first pioneered by Karl Terzaghi and laid down by Ralph Peck in his 1969 Rankine Lecture. This allows for (usually) predetermined modifications to be made during construction should performance depart from accepted tolerances.

1.1 ORGANIZATION OF CHAPTERS

Chapter 2 discusses the role of geology and geomorphology in modern geotechnical practice in Winnipeg. Winnipeg is situated in the flat terrain of the Canadian prairies on the bed of former glacial Lake Agassiz. The engineering properties of the soil deposits and surficial bedrock can vary considerably due to the depositional processes during glaciation and subsequent inundation by glacial melt water. Underlying the city is a stratified deposit of lacustrine clays and silts with unique engineering properties; in particular volume change potential and low residual shear strengths. The till deposits that underlay the lacustrine clay can be cemented or non-cemented, may contain cobbles and boulders and are often water bearing.

Deeper bedrock formations underlying the majority of the city consist of dolomitic limestone and dolomite and at great depth, basement rocks made up of granite and gneiss. The quality of limestone bedrock, in particular near its upper surface, can vary significantly and last, but certainly not least, is the potential for variability in groundwater levels and quality. This geological variability presents challenges to the practicing engineer in the design and construction of engineered works, in particular shallow or deep foundations. The engineering problems associated with geological variability, in particular as they relate to foundation design, were reported as early as 1926 (Fosness 1926) and were included in a report by the Committee on Foundations in Winnipeg, Manitoba (1937). Before this, considerable understanding was gained during the failure and righting of the Transcona Grain Elevator in 1913 (Figure 1-1) (Blatz and Skaftfeld 2002) and the construction of the Shoal Lake Aqueduct from 1914 to 1919 (Figure 1-2) (Robinson et al 2006).

Figure 1-1 Elevator binhouse after failure (c.1914). *The foundation failure and righting of the Transcona Grain Elevator is recognized as a truly remarkable case history made famous by its collapse after bearing pressures exceeded the limiting shear resistance of the clay foundation. The structure was successfully righted and remains in service today (photo courtesy of Bill Parrish Sr.).*

Figure 1-2 Construction of the Shoal Lake Aqueduct (c.1916). *Designers were faced with nearly unimaginable variability in ground conditions along nearly 100 miles from Shoal Lake to Winnipeg through the harsh geological environment left by the remnants of glacial Lake Agassiz (photo courtesy of the City of Winnipeg).*

One notable characteristic of soils in Winnipeg is the irregular presence of silt strata from about 0.2 to 1 m in thickness and encountered at a depth of 0.5 to about 4 m. In early years, this stratum was referred to as a "*band of yellow clay"* (Fosness 1926) and a "*yellow strip"* (MacDonald 1937). The soil layer was considered to be at least a contributing factor in early foundation problems because of its tendency to shrink and become hard when dried, and swell and become soft when wet. The tendency for shallow foundations to shift was also recognized, as was the role of changes in soil moisture content and temperature. In the 1950s, a significant amount of engineering effort was given to better understand the cause and effects of volume changes in expansive clay soils and design and construct shallow foundations that could be compatible with the inevitable movements (Hamilton 1969, Baracos 1969). The results from much of this work have been incorporated in the Canadian Foundation Engineering Manual published by the Canadian Geotechnical Society.

Chapter 3 discusses the role of groundwater in geotechnical practice. In the Winnipeg area, we often think of groundwater as the deeper confined aquifers situated in the glacial till or limestone bedrock formations. While this is true, we cannot neglect the influence of groundwater in near-surface soil formations in the design and performance of natural and engineered works (Figure 1-3). Groundwater will almost certainly be the root cause of a problem or claim during a geotechnical engineer's career. While such problems cannot be avoided entirely, the experience of others can act as a very helpful guide.

Figure 1-3 Pier foundations for CN Rail Bridge. *Discharge into pier excavations for a rail bridge over the Red River Floodway reached 8,200 litres per minute accompanied by slumping of the excavation side slopes. Groundwater discharge was believed to be either from hydraulic fracturing of the clay or along the driven H Piles. Grouting was required to stop groundwater flow (from Render 1970).*

Chapter 4 examines good quality site characterization techniques and instrumentation. The early drive to a better understanding of the engineering properties of soils resulted in much greater emphasis being placed on the importance of characterizing site conditions through soil explorations, good quality laboratory testing, *in situ* testing and exchanging technical information. This was clear as early as 1947 during a conference held in Ottawa under the auspices of the Associate Committee on Soil and Snow Mechanics hosted by the late R.F. Leggett (NRC 1947). In attendance were many of the pioneers of geotechnique in Canada, names like Charles Ripley, Hugh Sutherland, Gordon McRostie, and Robert (Bob) Hardy. One of the first matters to be discussed was the status of soils laboratory equipment available in Canada. Of particular local interest to us in Manitoba was news that a laboratory was now available at the Manitoba Department of Public

Works in Winnipeg and a laboratory was in the process of being organized at the University of Manitoba.

Site investigations should lead to savings in costs of construction and/or improvements in reliability. Careful planning is essential. It is important to talk with someone having field experience and take the time to plan in detail each aspect of the field investigation, from sample intervals to instrumentation to backfilling requirements. The cost of an investigation is also a consideration; how can the cost be optimized without compromising good quality and sufficient information?

There are several different drilling methods available locally to carry out sub-surface investigations and alternatives that can be brought in from other regions (Figure 1-4). Local drilling contractors can assist in determining the appropriate drill and drilling method if provided with preliminary information on soil conditions and sampling requirements. Mobilizing the wrong drill rig to a site will undoubtedly lead to unexpected delays or could compromise the quality of the investigation. Perhaps an excavator or even a hand auger may be more appropriate choices, depending on the nature of the investigation.

With some exceptions, geotechnical projects must incorporate earth materials that are already present – material properties must be measured, not specified. Laboratory tests are used to obtain mechanical and physical properties of soil and rock samples and determine how they can best be used in design. Looking back to when the field of geotechnique was first being developed, it would appear that there was a greater emphasis on laboratory testing to understand problems and aid in design than there perhaps is today. The most obvious reason is a matter of economics - the cost of carrying out good quality laboratory tests can be outweighed by scheduling and budget constraints. However, it may also be as a result of having developed a comprehensive understanding of local soils and their engineering properties. That is, through local experience, a good understanding of the relatively consistent nature of local Lake Agassiz clay has been developed. In some cases however, the consequences of insufficient laboratory test data may lead to the use of conservative (and perhaps costly) values for design. Junior engineers should not underestimate the value of carrying out laboratory tests and ensuring they are carried out by qualified technical personnel and in accordance with accepted standards. Completing a design with data from poorly run tests is perhaps worse than designing with no information at all.

Figure 1-4 Auger drill rig operating off a gravel pad along the shoreline of the Red River in Winnipeg. *Track mounted drill rigs can access sites that would otherwise be inaccessible to truck mounted drills.*

Chapter 5 discusses design methods for foundations in Winnipeg. In the early days of our profession, most civil engineering designs were based solely on experience, judgement and general sets of rules. Today, there are two main philosophies for civil engineering design; *Working Stress Design* (WSD) and *Limit States Design* (LSD). With WSD, a structure is designed by considering its stresses in a "working" condition using a global safety factor approach. With LSD, the structure is designed by considering the stresses at both the *Ultimate Limit State* (ULS) - the situation where the structure collapses, and the *Serviceability Limit State* (SLS) - the situation where cracks appear or the settlement is unacceptable but the structure has not collapsed. A limit state can therefore be considered a condition beyond which a structure (or foundation), will no longer fulfill the function for which it was designed.

The principal difference between WSD and LSD design methods is how uncertainty is accounted for. For reasons that will be discussed in Chapter 5, the current trend in civil engineering is to use Limit States Design. While its application in some areas of geotechnical design is straightforward, in other areas, its application is unclear. Chapter 5 reviews some of these difficulties.

Chapter 6 deals with deep foundations to support moderately to heavily loaded structures. As such, it will not include detailed discussion of shallow foundations for lightly loaded structures and the well-established environmental effects on shallow foundations due to the expansive properties of Lake Agassiz clay in Winnipeg. A discussion of design practices for deep foundations is considered timely given the recent shift from Working Stress Design (WSD) towards Limit States Design (LSD). The historical development of local design methods is valuable, and perhaps necessary. The intent of this chapter is to compare the design methods, and where appropriate, suggest applications of LSD methods which may produce designs more comparable with those expected from WSD methods. In this regard, it is not the Author's intent to question or modify codified design methods and parameters, for example, resistance factors, but to look more carefully at the basis for which traditional allowable foundation capacities have been adopted and how these values (in particular nominal capacities) relate to both the ultimate and service limit states.

Chapter 7 outlines the technical and regulatory framework that governs riverbank engineering in Winnipeg. Considering that there are more than 240 km of river and creek banks within the City of Winnipeg, a local geotechnical practitioner will undoubtedly become involved in a project where slope stabilization measures are required to address riverbank movements. Because of local soil conditions, groundwater levels, and river fluctuations, many of our riverbanks are inherently unstable. This work has become a specialized engineering field where an understanding of river morphology, channel hydraulics, environmental design and even archaeology is important. There are also regulatory requirements that may apply and which may have both prescribed and expected compliances.

Beginning in the 1950s, geotechnical engineers began to characterize riverbank movements and evaluate the influence of soil strengths and groundwater. In the last ten years or so, the sophistication of computer models to evaluate riverbank movements has dramatically improved. Limit equilibrium stability analyses can now readily be coupled with groundwater models to better characterize variable stress states associated with vertical gradients. The evolution of stabilization measures has been rapid, with many new techniques developed in the last 25 years. The understanding of soil-structure interaction has also been greatly advanced through many years of performance monitoring and research carried out by the University of Manitoba (Figure 1-5).

Figure 1-5 Aerial view of River Road test site. *A full scale field test was carried out to determine the performance of rockfill columns for riverbank stabilization and optimize design parameters (from Thiessen 2010).*

The last major chapter, ***Chapter 8*** provides lessons learned, conclusions and recommendations for future work. It discusses how experience is the basis on which engineering judgement can be applied in practice - judgement is perhaps the most important element in successful practice but it is also one of the hardest qualities to develop. Judgement should not replace theoretical problem-solving; rather, theoretical solutions should be used to make more sound judgements along each step of the process of design through to construction. The temptation to make decisions based solely on theoretical applications or advanced models that assume idealized material properties should be avoided. Perhaps taking a step back to a time where rigorous solutions were only possible after hours of calculations and painstaking preparation of detailed design notes should be a part of a geotechnical engineer's practical experience. By necessity, this would require that greater attention should be paid to the use of judgement in assigning soil properties or predicting potential failure modes but also provide a better appreciation of the intricacies of modern tools which can be easily taken for granted.

2 GEOLOGY AND GEOMORPHOLOGY OF THE WINNIPEG AREA

Geology is the science which studies the solid features of the earth and is traditionally subdivided into physical geology and historical geology. ***Physical geology*** is the study of materials which form the earth and the many processes which act upon these materials both on and beneath the surface of the earth. ***Historical geology*** uses the principles of geology to understand the origin and development of the earth. An understanding of geology is important to the geotechnical engineer as almost every natural material we work with has basic physical and chemical characteristics determined by the physical geology of the region and its evolution over geologic time. An understanding of geology is also of critical importance in understanding natural hazards and methods to protect the public from these hazards.

Geomorphology is the study of landforms and the physical and chemical processes that created these landforms. The area of interest to the geotechnical engineer is often referred to as *Process Geomorphology* as this deals with the processes that change the landforms. These processes include wind, mass wasting, wave action, river erosion and weathering to name a few. All of these are important considerations in the practice of geotechnical engineering as they are mechanisms that create the landforms we work with and the irreversible processes that we endeavour to control through our engineered works. Areas of geotechnical practice that deal with geomorphologic processes and their control are perhaps one of the most challenging for the geotechnical engineer.

2.1 BEDROCK GEOLOGY

The lacustrine deposits and till within the Winnipeg region are underlain by carbonate sedimentary bedrock from the Palaeozoic era or more specifically, from the Ordovician period, making these formations almost half a billion years old. The three formations of Ordovician age underlying the Winnipeg region are the ***Stony Mountain, Red River, and Winnipeg Formations*** (Baracos and Kingerski 1998). These formations dip to the southwest as shown on Figure 2-1. The Ordovician formations are in turn underlain by Precambrian basement rocks made up of granite and gneiss. The Precambrian basement rock is generally too deep to be of significance in local design and practice.

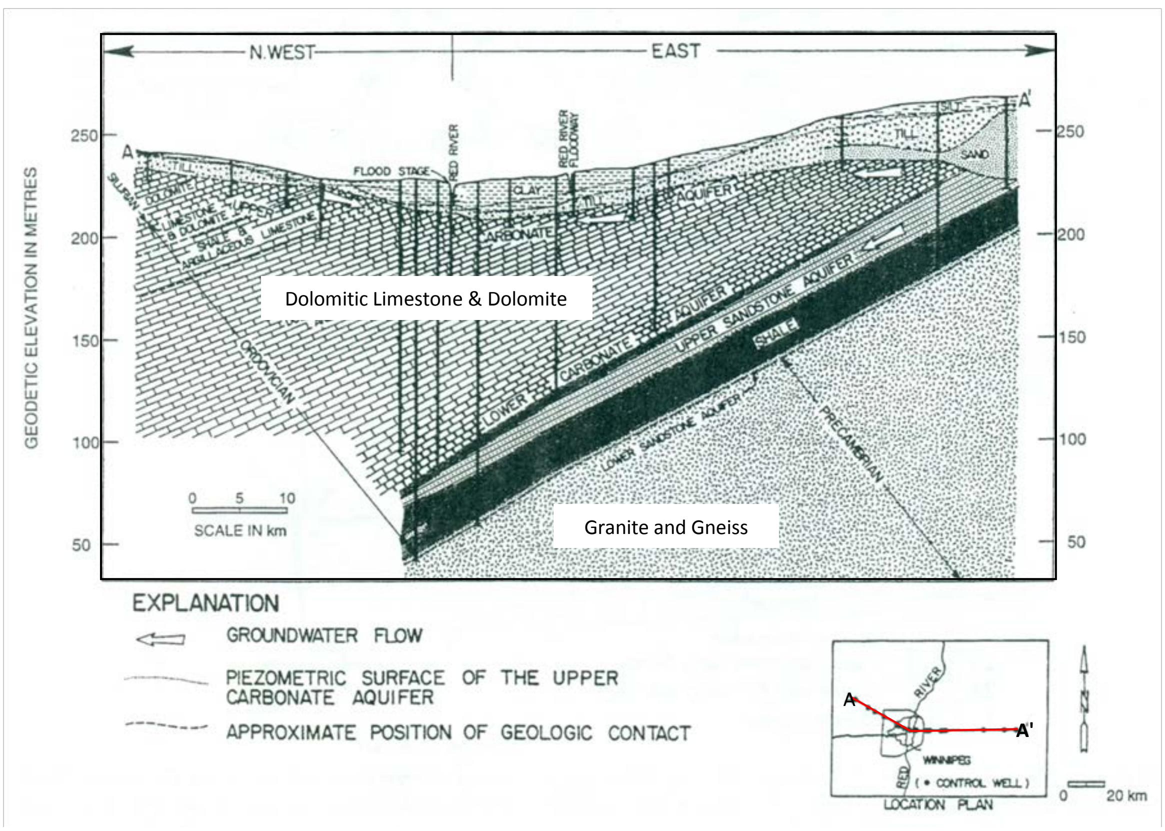

Figure 2-1 Cross section A-A' through Winnipeg showing the Ordovician age carbonate bedrock. *The predominantly carbonate rocks were deposited in shallow seas that had inundated the Precambrian basement (modified from Render 1970)*

Manitoba Energy and Mines published an excellent summary of dolomite resources in southern Manitoba (Bannatyne 1988). The report includes detailed descriptions of the engineering properties of the bedrock within the Winnipeg region (Figure 2-2). The shallow bedrock unit is often categorically referred to as ***limestone*** although it mainly consists of dolomite or dolomitic limestone, a type of limestone composed of both magnesium and calcium carbonates. Quarried dolomitic limestone from the region close to Tyndall Manitoba is often referred to as ***Tyndall Stone*.** It is important to understand that significant variability exists in the physical and chemical properties of carbonate bedrock with the differences being a result of transgressions of the Ordovician seas (Bannatyne 1988).

Rock from the Stony Mountain and Red River Formations is used to produce aggregate and building stone and is often associated with the design and construction of deep foundations (for example, rock-socketted caissons). A third and much younger rock from the Jurassic Period (about 150 million years old) known as the Amaranth Formation consists of sediments deposited in eroded channels in the older carbonate rocks. The Winnipeg Formation, consisting of sandstone and shale, also underlies the Winnipeg region but is generally too deep to be of consequence in local practice. It does occur however at shallower depths about 60 km east of Winnipeg. Each of the major carbonate bedrock formations is divided into ***members*** of varying texture and composition as shown on Figure 2-2 which also shows the locations of these bedrock members within the Winnipeg region and shows project locations referred to in this thesis.

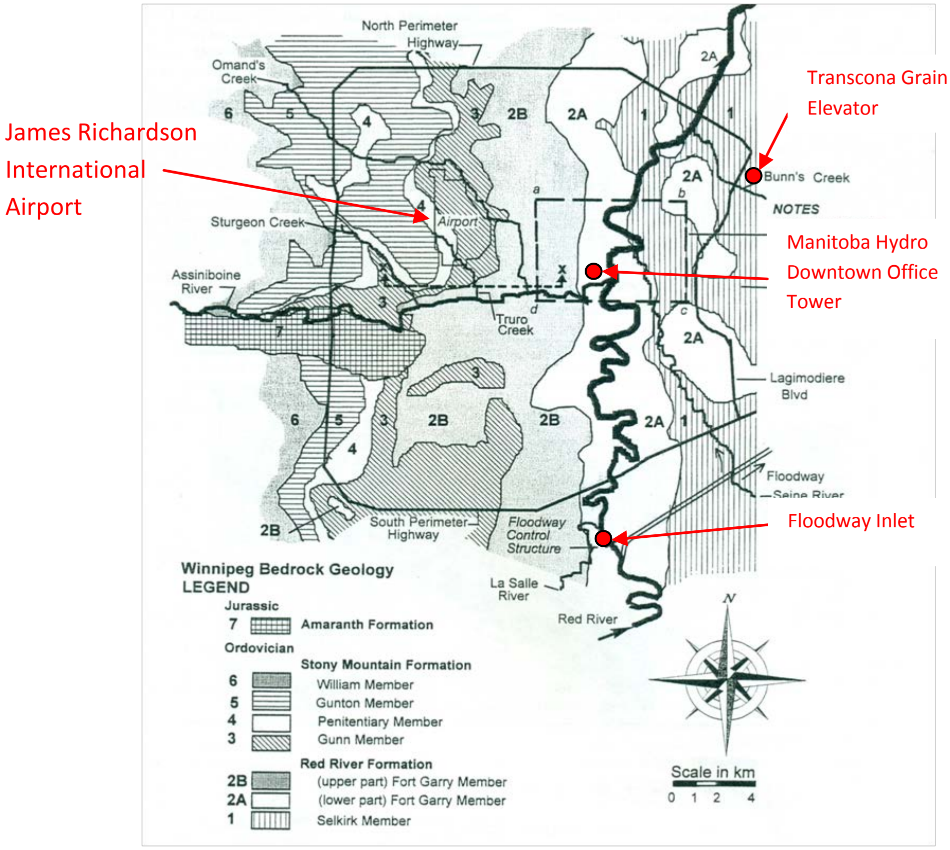

Figure 2-2 Bedrock Geology in Winnipeg Region. *There are significant differences in the composition and engineering properties of the carbonate bedrock formations. These differences can be encountered over relatively short distances, often within the confines of a construction site (modified from Baracos and Kingerski 1998).*

Carbonate deposition began with the Red River Formation which has been divided into the Dog Head, Cat Head, Selkirk and Fort Garry Members (deepest to shallowest). It is the upper two members (Selkirk and Fort Garry) that are typically encountered in engineering works. An increase in clastics composed of cemented fragments of older rocks, resulted in the deposition of the Stony Mountain Formation which consists of the Gunn Member (red shale), the Penitentiary Member (argillaceous dolomite), the Gunton Member (dolomite with variable argillaceous content), and the Williams Member (argillaceous and sandy dolomite).

2.1.1 Red River Reformation

The ***Selkirk Member*** is found within the eastern portion of the Winnipeg region as shown on Figure 2-2. North of Winnipeg, the bedrock is close to the ground surface and is the source of Tyndall Stone, a high quality dolomite used for building stone. The stone is obtained from an active quarry in Garson, Manitoba operated by Gillis Quarries. Several quarries in East Selkirk and north of Lockport have largely been depleted. While the overall quality is good, there is often varying hardness associated with dolomite-rich and limestone-rich mottles[1].

The ***Fort Garry Member*** consists of Upper Fort Garry and Lower Fort Garry layers which trend north-south along and west of the Red River as shown on Figure 2-2. Because it passes beneath the downtown area, many of the deep foundations in the City of Winnipeg are supported on the lower layer of the Fort Garry Member (Figure 2-3). Isolated occurrences of Cretaceous shale, silica sand and/or black lignitic silty material, known as the Swan River Group, may be encountered in solution cavities or channels in the Upper and Lower Fort Garry Members in the downtown area.

The Fort Garry Member subcrops north of Winnipeg where several quarries are located, for example Mulder Construction. Within these quarries, the upper and lower layers are typically good quality dolomite separated by a reddish coloured argillaceous (shale) marker bed.

[1] Blotches of different shade or colour

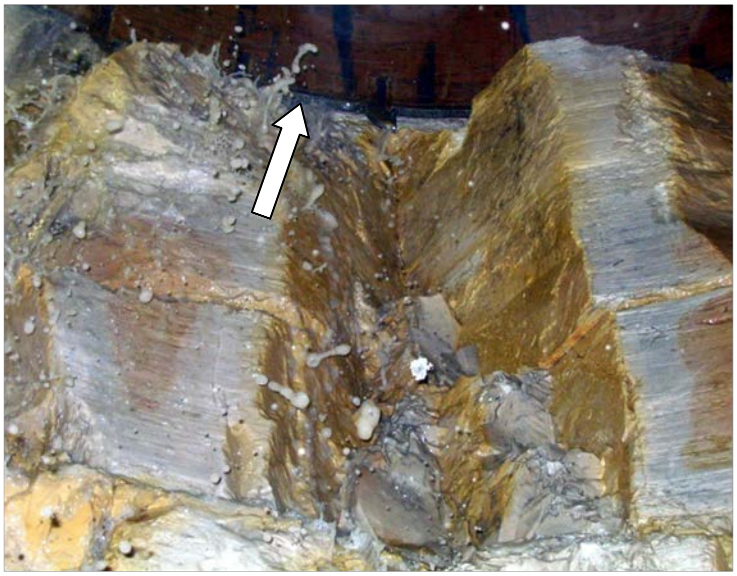

Figure 2-3 Rock socketted caissons for Manitoba Hydro's downtown office tower. *The sockets extend into the Fort Garry Member - The dolomitic limestone can be seen just below the steel casing fitted with cutting teeth (see arrow). The surface layer of the bedrock is heavily fractured and water bearing. These sockets were extended into sound bedrock and dewatered for inspection.*

2.1.2 Stony Mountain Formation

The Stony Mountain Formation consists of the Gunn, Penitentiary, Gunton and Williams Members. The quality of the rock ranges from poor to excellent and therefore provides material for a variety of engineering applications. It has been the primary source of raw material for lime and crushed aggregates for many years, with the higher quality portions of the formation providing high quality dolomite for applications including road construction, rockfill columns and rip rap.

The ***Gunn, Penitentiary, and Williams Members*** contain the poorest quality material. The Gunn Member consists of red to purple calcareous shale interbedded with thin layers of limestone while the Penitentiary and William Member consists of argillaceous dolomite. The Gunn, Penitentiary and Williams Members are encountered in the western half of the City of Winnipeg and in quarries north of the city that produce lower quality aggregate for less intensive engineering applications such as surface gravel and back lane construction (Figure 2-4). The Gunn member was encountered during exploration work for the construction of the James Richardson International Airport.

Figure 2-4 Stony Mountain quarry north of Winnipeg. *The Stony Mountain Formation is visible on the north wall of the quarry. Visible are the purple limestone and shale of the Gunn Member (fieldbook in the middle of the layer) and the yellowish Penitentiary Member*.

The Gunton Member is found along the western limits of Winnipeg extending north through the Stony Mountain-Stonewall region and Gunton, Manitoba. It is recognized as the source of high quality dolomite for applications such as aggregate for road and highway construction, rock columns, railway ballast and potentially for concrete and asphalt aggregate.

2.2 ENGINEERING SIGNIFICANCE

All rock ultimately weathers and the rate of weathering depends on the mineralogy and physical properties of the rock. The deterioration of crushed rock aggregates is influenced by a number of these properties, including porosity, strength and frequency of wetting, drying, freezing, and thawing. Given the large volume of aggregates produced from quarried limestone for a variety of applications (for example, rockfill columns) and the dependency on good quality rock for deep foundations (for example rock-socketted caissons), geotechnical engineers should be familiar with the different bedrock units and, most importantly, the physical properties of these units. It is also important to recognize that significant variability in the bedrock often occurs over short distances, often within the confines of a quarry or construction site.

The physical properties of carbonate bedrock are variable, with the largest differences expected between the different members. Colour should only be used to provide a preliminary estimate of the bedrock type and care should be exercised when rejecting material based on colour alone. It has been the Author's experience that pinkish coloured aggregate can be of good quality, while yellowish and reddish colours are often (but not necessarily) indicators of poor quality material. With experience, a geotechnical engineer should be able to make a visual assessment of the overall aggregate quality to at the very least, determine the testing requirements. It is advisable to inspect the aggregate at the quarry where it is being produced so that the nature of the bedrock can be visually assessed and any corrective measures addressed before material has been delivered to site.

The physical properties of the bedrock members have been reported by Manitoba Energy and Mines (Bannatyne 1988). Information from this report is summarized in Table 2-1. The tests carried out include Los Angeles Abrasion Loss, soundness loss, bulk specific gravity, absorption, porosity and unconfined compressive strength. The values represent the average of tests and the average of tests between aggregate sizes for soundness loss.

TABLE 2-1 Physical Properties of Bedrock Members (from Bannatyne 1988)

Physical Property	BEDROCK UNIT					
	Red River Formation			Stony Mountain Formation		
	Selkirk Member	Lower Fort Garry Member	Upper Fort Garry Member	Gunn Member	Penitentiary Member	Gunton Member
Bedrock Type	**Dolomitic Limestone**	**Micritic Dolomite**	**Vuggy Cherty Dolomite**	**Argillaceous Dolomite**	**Argillaceous Dolomite**	**Dolomite**
LA Abrasion Loss (%)	49.8	32.5	32.7	45.7	58.0	28.7
Soundness Loss (%)	12.3	28.4	21.9	48.2	100.0	17.2
Bulk Specific Gravity	2.40	2.64	2.52	2.56	2.38	2.61
Absorption (%)	3.75	2.2	3.1	2.6	5.6	2.1
Porosity (%)	9.1	5.9	7.7	6.6	13.7	5.6
Unconfined Compressive Strength (MPa)	35	88	61	No data	29	92

Although Los Angeles (LA) abrasion loss is commonly used by industry to measure aggregate quality, it is more a measure of fracture toughness and does not provide an indication of cold climate stability. There are two ASTM Standards used for the LA abrasion loss test; ASTM C131 for small size coarse aggregates and C535 for large size coarse aggregates. There is no consistent relationship between the two test methods (for the same material), although losses may increase with reduced aggregate size. Because of these inconsistencies, comparative measures are difficult, even when the same quarry source is used. It is the Author's opinion that it may be advisable to consider the exclusive use of one test method, and perhaps one grading size, irrespective of the maximum particle size specified.

Degradation from repeated freeze-thaw cycles results in gradual disintegration of the aggregate and the generation of fines that can contribute to frost heave and reduced drainage properties. Because the freeze-thaw test takes considerable time and requires specialized testing facilities, the soundness test is used as an alternate test method to provide an indication of the aggregate's susceptibility to breakdown through mechanical weathering by simulating water's expansion upon freezing. In general, the higher the soundness loss, the less durable will be the material, for example in a pavement structure. Hence the amount of water a rock can absorb (water absorption) is an important physical property.

Similar to the LA abrasion test, soundness loss test results are usually different for fine and coarse aggregate; typically with higher losses for the finer aggregate (usually a weighted average is used). It is also important to recognize that test results will depend on the type of salt used for the test (either magnesium or sodium sulphate). Higher losses can generally be expected when magnesium sulphate is used. Since the precision of this test method is questionable, the results should not generally be used for outright rejection of the material without confirmation of material properties from other test methods (for example, LA abrasion). The Author considers that specifications for crushed limestone should include a maximum soundness loss value as this may be the property that best determines the physical performance of the aggregate.

Work by Franklin (2003) looked at the relationship between physical properties of quarried riprap relative to potential degradation caused by freezing and thawing. The author has combined results from this work (off-white limestone samples only) with the work by Bannatyne (1988), and aggregate testing carried out by industry (Author's personal file) as shown on Figure 2-5. The results show there is a strong relationship between bulk specific gravity and water absorption[2], with absorption decreasing with increasing bulk specific gravity. In general, higher absorption is an indication of higher permeability and porosity[3].

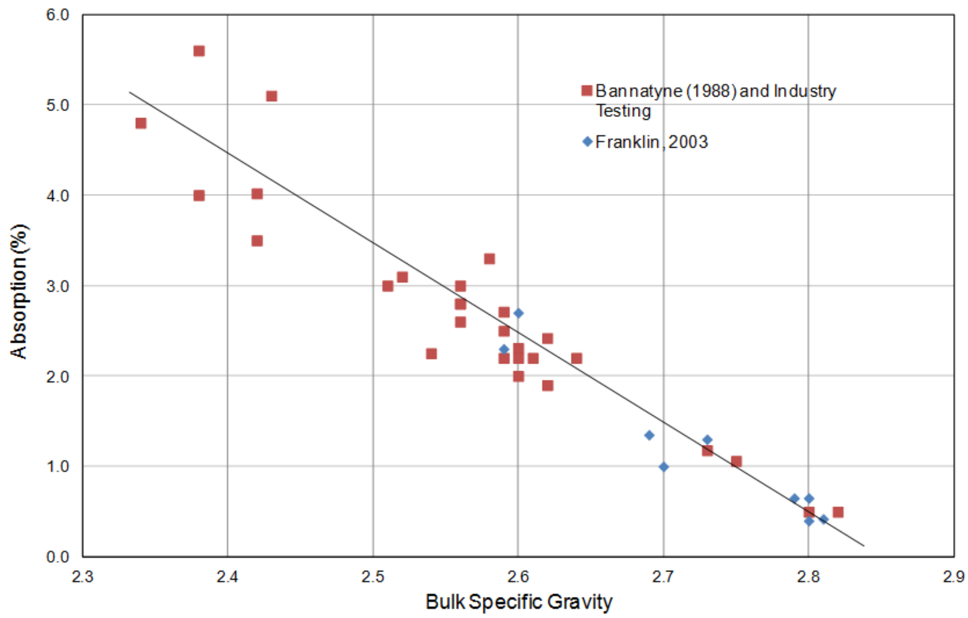

Figure 2-5 Apparent relationship between bulk specific gravity and water absorption

Further to this evaluation, the results of testing for a number of physical properties have been compared with each other to evaluate possible relationships. Figure 2-6 suggests that a relationship exists between absorption and soundness loss showing that soundness losses will be about 25% or lower if the absorption is less than about 2.5%, a value which could therefore be considered for inclusion in specifications, or alternatively, could be used as an indicator when evaluating and approving crushed limestone. This would be

[2] the amount of water present in rock pores as a percentage of dry mass
[3] the measure of void space in rock

useful method to predict soundness loss for the purposes of conditional approval or perhaps the basis for rejection of material pending the results of a soundness loss test that takes up to 2 weeks to complete (the absorption test can be completed in 1 to 2 days).

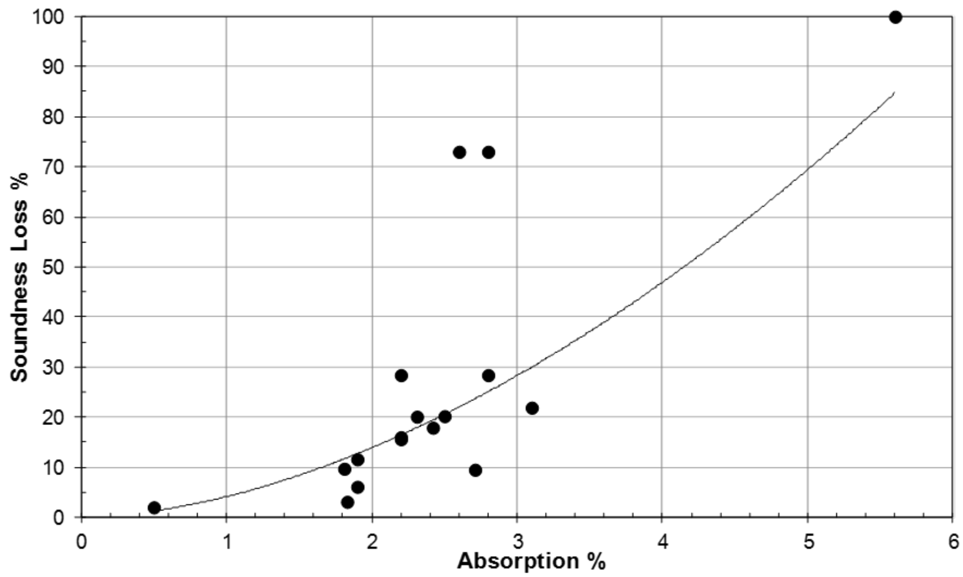

Figure 2-6 Apparent relationship between absorption and soundness loss

The Iowa Pore Index Test was originally developed by the Iowa Department of Transportation to test aggregate durability in concrete pavement - the work by Franklin (2003) has also shown this test to be much quicker than freeze-thaw testing but with comparable results for tests carried out on limestone aggregate. Further work to study the relationship between the Iowa Pore Index Test and soundness testing on limestone would be of value.

Similar relationships compiled by the Author appear to exist between bulk specific gravity and soundness loss (Figure 2-7), bulk specific gravity (SG) and LA abrasion loss (Figure 2-8) and bulk specific gravity and unconfined compressive strength (Figure 2-9). While caution should be exercised if using these relationships to predict other physical properties, they may help a geotechnical engineer in carrying out a more meaningful evaluation of the suitability of bedrock or aggregate for construction purposes.

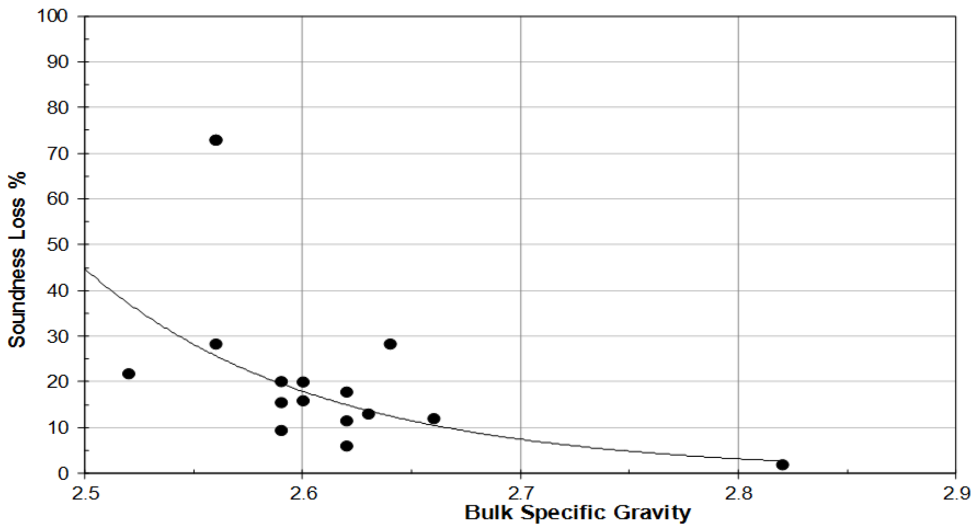

Figure 2-7 Apparent relationship between bulk SG and soundness loss

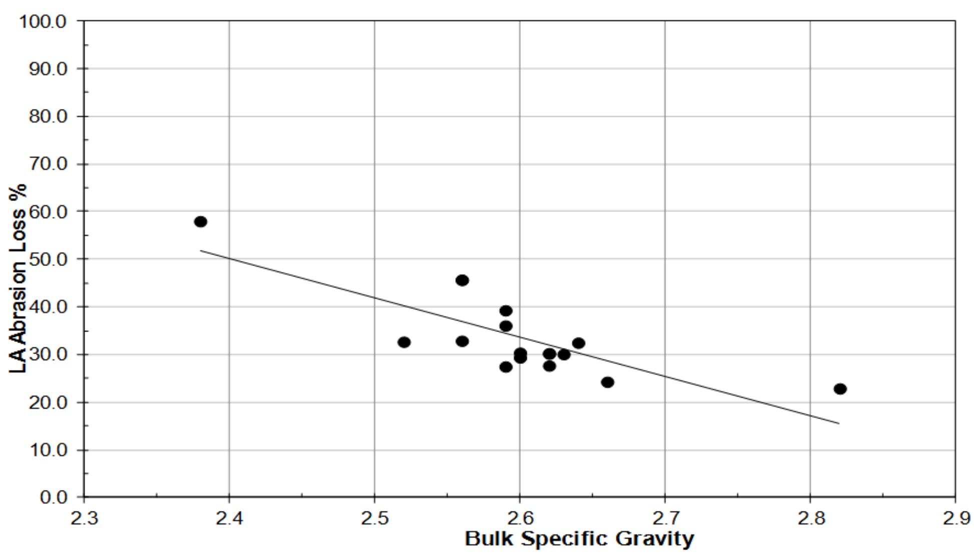

Figure 2-8 Apparent relationship between Bulk SG and LA Abrasion Loss

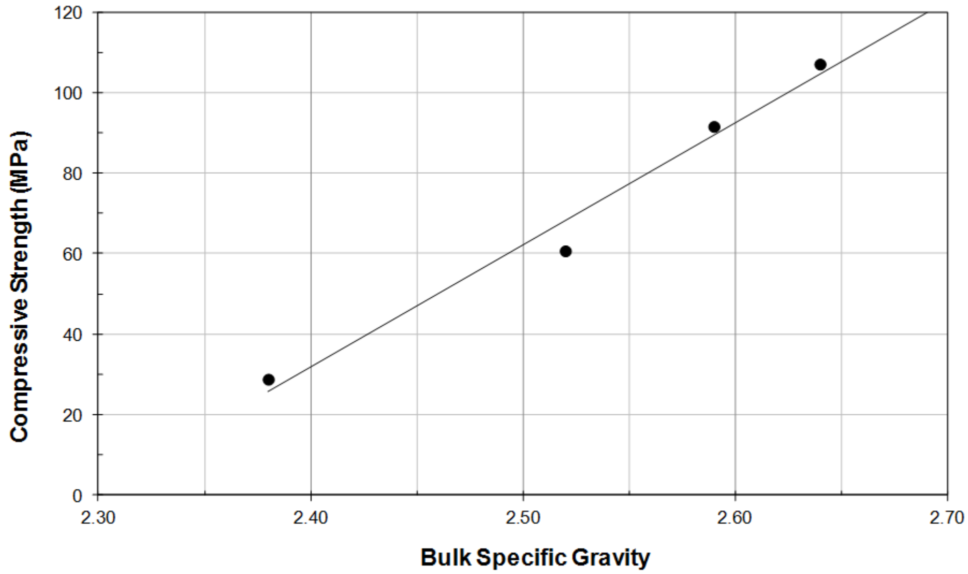

Figure 2-9 Apparent relationship between bulk SG and compressive strength

Of particular importance to local practice is the relationship between LA abrasion and soundness loss as many specifications only provide a minimum value for the former. Figure 2-10 shows the relationship between soundness loss and LA abrasion loss from a variety of bedrock quarries with bedrock formations in the vicinity of Winnipeg. There is a large cluster of test results showing LA Abrasion losses from 20 to 40% have corresponding soundness losses from about 2 to 28%. The two test results with soundness losses greater than 70% represent samples from the Gunn (73% loss) and the Penitentiary Members (100% loss). Clearly, the lower the LA abrasion and soundness losses, the better quality the material.

Before specifying the physical properties of a limestone aggregate, it is necessary to consider the engineering application for which it will be used. Recognizing the variability in material quality from local quarries and the finite amount of high quality material (dolomite in particular) that is available, a range of soundness values may be appropriate, with the maximum specified value based on the intended application. In this regard, the Author considers that a maximum allowable soundness loss of 18% should be specified for crushed limestone where high quality aggregates are required, for example for regional streets and rock columns. In cases where lower quality material would be acceptable, for example low volume residential streets and back lanes, a maximum soundness loss of 25% is recommended. In both cases however, the maximum LA Abrasion Loss should be 35%. This approach may help industry manage their quarry operations more economically and extend the availability of better quality material without compromising the design life of engineered works.

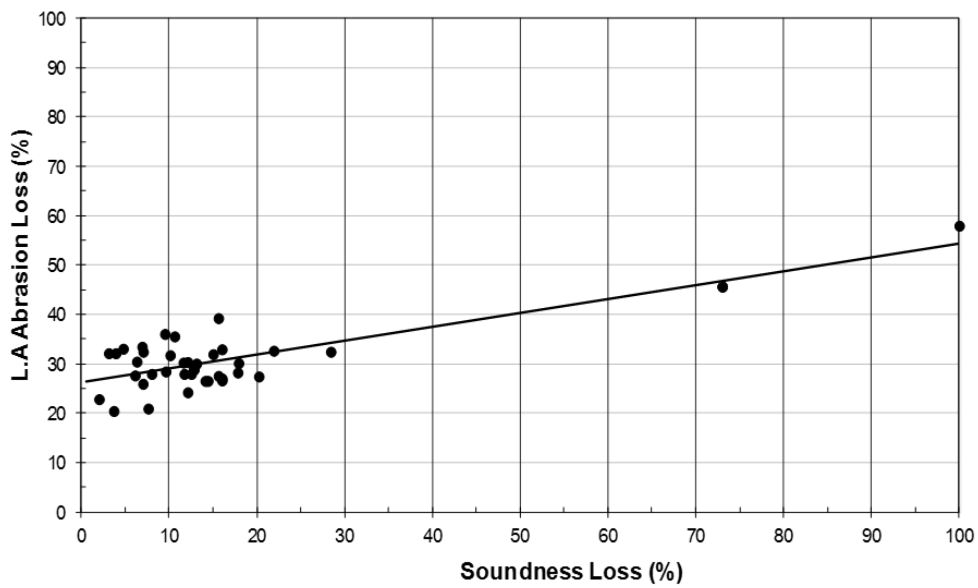

Figure 2-10 Apparent relationship between soundness loss and LA abrasion loss

2.3 QUATERNARY GEOLOGY

Early in the Quaternary Period, continental ice sheets covered most of Canada and the northern United States, during which time, till deposits blanketed most of the bedrock in the Winnipeg area. When the last ice sheet retreated about 12,000 years ago, glacial Lake Agassiz was formed and sediments brought into the lake were laid down as glaciolacustrine (glacial lake) deposits of clay and silt. It is important for geotechnical engineers to have a good understanding of the engineering properties of these materials for the design and construction of engineered works.

2.3.1 Till Deposits

Where present, till deposits overlying bedrock in the Winnipeg area are generally encountered at depths ranging from 3 metres in the northwest to 18 metres in the east (Render 1970). The layer is occasionally absent but generally of substantial thickness of up to 10 metres (Baracos and Kingerski 1998). The till is of varying consistency with the dense to very dense portions of the deposits being a basal till[4]. The upper horizon of the till deposit may be considerably softer, likely an ablation till[5].

The till is a heterogeneous mixture of grain sizes ranging from small fractions of clay to gravel sizes, with the predominant fraction often being silt. Cobbles and boulders are often found in the till and their presence can be of significant consequence in the design and construction of deep foundations. Care must be taken when interpreting power-auger refusal, especially if small diameter augers are used for subsurface exploration. In some cases, power-auger refusal may be reached prematurely on a cobble bed or boulders rather than on the dense till strata which may be several metres deeper. For this reason, larger augers such as used on a piling rig, are often used for foundation investigations as they can generally penetrate through cobbles and small boulders to reach the dense till material.

[4] Often locally referred to as "hardpan".
[5] Often locally referred to as "putty till".

Readers should note that the different till layers may be difficult to distinguish without the aid of results from water content tests. In this regard, water contents in the till generally decrease with increasing depth and correspond to increases in strength. The upper ablation till (if encountered) typically may have water contents ranging from 10 - 15% while the denser basal till will typically have water contents in the range of 7 - 10%. The water content of the deeper very dense basal till can be as low as about 5%. Care must be taken when sampling to avoid mixing wet material from shallower depths with drier material. When taking 'disturbed' samples, it is good practice to carefully waste material from the outside of the auger sample that may have been wetted , and collect only samples of drier till from near the auger stem.

Unit weights and strength of the basal till may approach that of concrete and therefore this soil unit has excellent strength/deformation properties from a foundation perspective. It is important, though, to confirm that bedrock below the till is intact, not karstic. Standard Penetration blow counts (N) in the very dense till associated with these values are typically greater than 75 blows per 300 mm and often reach refusal before the specified test penetration depth of 300 mm. Unconfined compressive strengths ranging from 3.4 - 3.6 MPa have been reported for very dense tills with a moisture content of about 5% (Kjartanson 1983). Young's moduli typically range from 170 to 240 MPa (Kjartanson 1983) although pressuremeter testing has yielded values as high as 450 MPa (personal communication, Dr. R.M. Kenyon). It is important to recognize that while the till may be very dense upon initial exposure, it can quickly become softened when in contact with water.

2.3.2 Glacial lake Deposits

Deposits of glacial lake clays and silts of varying thickness and composition overlie the glacial till deposits throughout most of the Winnipeg area. The composition of these lakebed deposits ranges from predominantly high plastic clay to silty clay to predominantly silt. The upper 3 metres is often referred to locally as the ***Upper Complex Zone***. Within this zone, the clay typically contains higher silt fractions and often distinct horizontal layers of silt similar to that seen in Figure 2-11. This layer, referred to in historical papers as *the yellow strip or the streak of yellow clay*, was recognized as running across the entire Winnipeg region, except along riverbanks where it has been washed out and replaced with river silt (Fosness 1926). The problems encountered when

Figure 2-11 Basement excavation. *The Upper Complex Zone often contains silt deposits of significant thickness and aerial extent. Here, the tan coloured silt layer can be seen across the side of a basement excavation in south Winnipeg. Also of note are large shrinkage cracks in the highly plastic clay after only a few days of exposure.*

placing foundations on this unit are well known and in one case, the layer was reported to being the curse of many of the old foundations in this district (Fosness 1926).

In decreasing occurrence, the predominant mineral composition of the lacustrine clay generally consists of montmorillonite (a member of the smectite family), illite, kaolinite and some mica (Graham and Shields 1984). The lacustrine clay typically trends from brown to grey (sometimes referred to as blue) at depths of approximately 4 to 6 metres. Within this depth range, the brown and grey clays often appear mottled, making it difficult to identify a distinct contact between the two colours. It is believed the colour change is due to oxidation (the brown clay being oxidized) as there is no obvious change in mineralogy, clay content or plasticity (Graham and Shields 1985). Wicks (1965) suggested that upper layers of varved clays may aid in oxidation while the massive grey clays at depth are still in a reducing environment that inhibits oxidation, or even that the oxidation may still be an active process. In the Author's opinion, while the colour change from brown to grey may be an indication of the depth of historical oxidation, it is not a reliable representation of the modern day depth to the water table, which in all likelihood, which may be shallower.

Along the shoreline and shallow near-shore waters of glacial Lake Agassiz, waves would have eroded ice-laid drift material (Ehrlich 1953), transporting the finer grained material to the deepest part of the lake (in the vicinity of Winnipeg, the depth of water is estimated to have been 550 to 600 ft) where it was deposited across the till mantel.[6] Ordinarily, this method of deposition would result in distinct varves or stratification of the sediments. Marbling (irregular swirls) in the clay (in particular the grey clay) however, suggests that the clay may have been mixed, possibly by the mechanical action of iceberg scouring (Baracos 1976). It has also been speculated that wave action in shallow water away from the ice front may have reached the surface of the sediments thus preventing the varves from forming (Wicks 1965). This effect would likely have blurred the horizontal

[6] The transportation of drift in large rivers leading into Lake Agassiz and the consequent formation of silts in proximal deposits and clays in distal deposits is another possibility.

laminations which may have otherwise have occurred during deposition of the clay. The grey clay is often described as being "massive" (Wicks 1965).

The brown clay may be somewhat stratified, possibly as wave action diminished as the lake receded and its depth decreased (Ehrlich et al 1953). Besides being stratified, the near-surface brown clay is often heavily fractured and fissured creating a blocky or nuggety structure. The size of these nuggets is small, generally ranging from a few millimetres near the ground surface to perhaps a centimetre, with each nugget exhibiting a noticeable glossy surface. This structure gradually diminishes with depth and is almost non-existent perhaps 2 or 3 m from ground surface. Larger vertical or sub-vertical fissures are seen at depths down to 6 m. Environmental effects such as weathering and desiccation are most likely responsible for the fissuring.

The engineering significance of the blocky and fissured structures is important, particularly in excavations that may expose the clay to such environmental effects for the first time. The development of micro-fracturing may allow the ingress of water and subsequent swelling and softening. More importantly, the shear strength of the clay may be significantly reduced in excavations where the effective overburden pressure has been reduced and the material may take on the properties of a cohesionless granular aggregate (Thompson and Kjartanson 1985). It is good practice to cover the exposed excavation face with plastic sheeting to minimize environmental effects.

The origin of the calcareous silt layer within the brown clay unit is often a point of considerable discussion. If laid down in water, it may be a lacustrine deposit, an alluvial deposit or perhaps a fluviolacustrine deposit, having been deposited in shallow remnant ponds of Lake Agassiz by the Red River and its tributary rivers (Ehrlich at al 1953). The origin of the silt layer is perhaps best described by Wicks (1965) who offers the explanation of a wind-borne deposit (loess) as the most logical explanation given the structureless bed of very uniform composition. Work by Baracos (1977) however, shows that the silt layer exhibits distinct varves or layering, perhaps more indicative of a water-borne deposit. The eventual lowering of glacial Lake Agassiz was not a steady process, but rather, the lake may have periodically drained and then refilled (Bluemle 1975). Therefore, during the period when the lake was drained, it is possible that wind borne and alluvial deposits were laid down as silt and subsequently covered with the last thin layer of lacustrine (or perhaps fluviolacustrine) clay when the lake partially refilled. Perhaps there is not one unique process, but rather a series of processes that occurred during changes that took place as the last of the ice sheet melted.

The grey clay layer has in some cases been subdivided into upper "grey clay" and a lower "grey plastic clay" units, however, the results of X-ray diffraction and scanning electron microscopy does not support such distinction. Rather, the slight differences in engineering properties such as plasticity may be due to changes in clay/silt content or to inclusions of non-plastic veins and inclusions (Baracos 1977). These inclusions are typically about 10 mm in diameter and predominantly silt. The origin of these inclusions (sometimes referred to as clasts) is uncertain but they are believed to have been either dropped into the clay by icebergs (ice-rafted) as frozen particles or as lithified or cemented particles, for example, limestone which subsequently disintegrated. The near absence of these inclusions at shallow depths within the clay suggests a retreat of the glacier front during the latter stages of deposition of the lake bed deposits. At depth, the frequency of the silt clasts increases and may appear as studded bands near the till contact (Trainor 1982). Limestone (and occasionally granite) pebbles are also found within the grey clay, in particular as the underlying till is approached.

Of particular interest are inclusions of white material which are often referred to as sulphate or gypsum inclusions. This is of practical importance given the potential for sulphates in solution to cause significant deterioration of concrete. Gypsum in fact is calcium sulphate so either term used in describing inclusions would be an indication of a potentially aggressive environment for concrete in contact with soil. While testing for soluble sulphates in soil can be undertaken, it is local practice to assume they are present and accordingly, specify the use of sulphate resistant cement.

There is also evidence that gypsum is present in much of the lacustrine clay in the form of calcium sulphate precipitated as cementation bonding at interparticle contacts. The gypsum cementation is seen in oedometer tests as apparent overconsolidation (Man and Graham 2010, Man et al. 2011), with well-defined yielding behaviour (Graham et al. 1983), and strain-softening (brittle) stress-strain relationships. The latter requires analyses of slopes and embankments to use ‘post-peak’ (or normally consolidated) strengths and not ‘peak’ (or overconsolidated) strengths (Rivard and Lu 1978).

A typical clay profile will yield water contents that increase with depth, from around 30% in the upper 3 m to about 50% or higher at the base of the layer above the till contact and which typically fall midway between the plastic and liquid limits, although exceptions do occur (Figure 2-12). The lacustrine clays are subject to considerable volume changes with changes in water content, and this potential behaviour should never be overlooked, especially in the upper complex zone where large variations in water content are common. These volume changes can result in heave or settlement as large as 150 mm and occasionally greater, leading to serviceability problems with foundations, pavements and utilities. Another important consideration is the compressibility of the clay when loaded beyond its preconsolidation pressure. A notable factor in the assessment of consolidation settlement is the propensity for the upper horizon to behave as if moderately to heavily over-consolidated, a property that is exploited to keep foundation settlements to within a tolerable range.

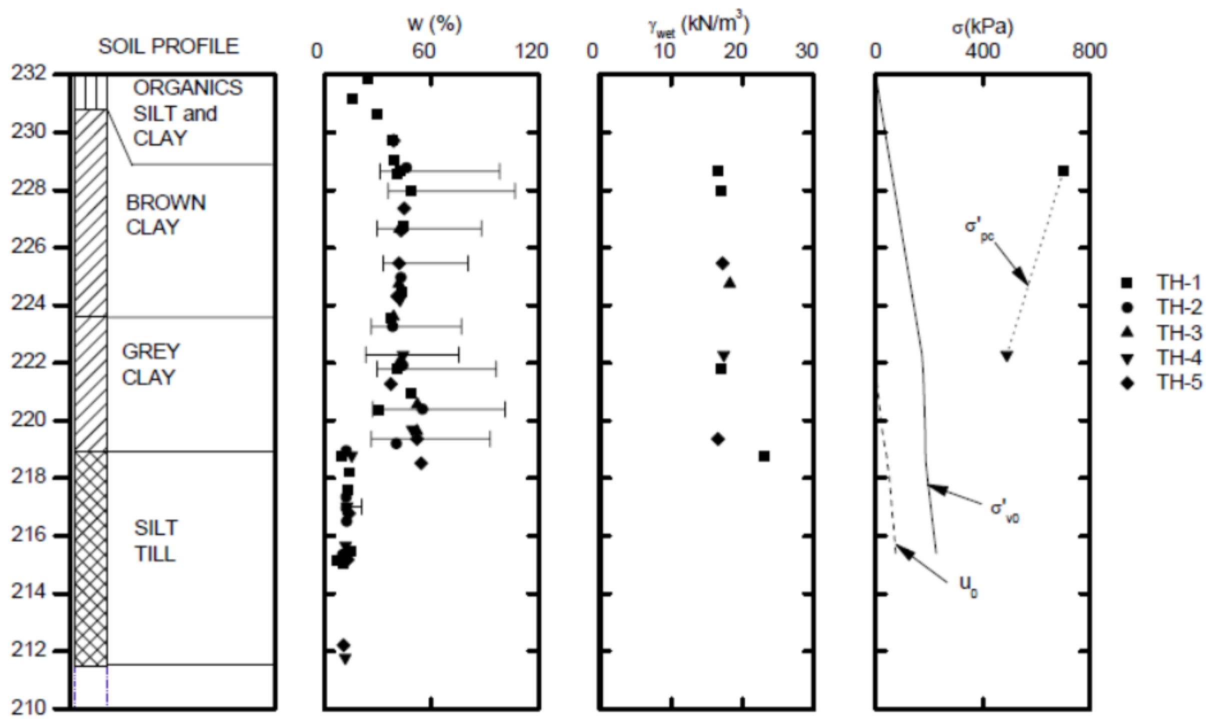

Figure 2-12 Typical stratigraphic profile and laboratory test results (from Thiessen 2010)

Because water contents are significantly higher than the plastic limits, compaction often requires that the material be air dried. Of significant practical value (and often overlooked), is determination of the liquidity index as a way of assisting contractors to determine if the consistency of the clay may hinder the operation of heavy equipment. The liquidity index (I_L) is a value that relates the field water content of a soil to the Atterberg Limits for the soil, specifically the plastic and liquid limits. A liquidity index of 0 means the soil is at the plastic limit while a liquidity index of 1 means the soil is at the liquid limit. Values of I_L between 0 and 1 then provide an indication of the consistency of the soil, approximate bearing capacity, and expected workability. In this regard, the guidelines provided in Table 2-2 are often used locally. Mishtak (1962) related the depth of excavation where the clay became too wet to permit the use of rubber tired equipment on the Floodway excavation as 7.5 m. From laboratory test data for the Floodway project, the liquidity index at this depth is about 0.5.

Table 2-2 Engineering Significance of Liquidity Index

Liquidity Index	Consistency	Allowable Bearing Capacity (kPa)	Comment Relative to Workability
0 - 0.25	Stiff	96 - 192	
0.25 - 0.5	Medium Stiff to Firm	48 - 96	
0.5 - 0.75	Soft	24 - 48	
0.75 - 1.0	Very Soft	< 24	
< 0.4			Use of Rubber Tired Scrapers Possible
0.4 - 0.5			Use of Rubber Tired Scrapers Questionable
> 0.5			Use of Rubber Tired Scrapers Not Possible.

As the till is approached, the clay may contain numerous till inclusions as evidenced by water contents ranging from 15 to 25%. The layer, if encountered, is generally very soft and easily penetrated during drilling, although squeezing and sloughing of the borehole may prevent extending the augers to greater depths. The layer may be locally referred to as a "till transition zone" or "inter-till"

Undrained shear strengths measured from unconfined compression tests are generally higher within the upper active zone, typically in the order of 75 to 100 kPa. Below a depth of about 4 to 5 metres, strengths typically decrease approximately uniformly with increasing depth. As the underlying till layer is approached, strengths are typically in the order of 40 kPa but may be as low as 25 kPa. The typical pattern of higher undrained shear strengths near the surface and lower shear strengths at depth reflects weathering near the ground surface and decreasing overconsolidation ratios to approximately normally consolidated conditions near the bottom of the deposit. They may also reflect artesian ground water conditions (and therefore low vertical effective stresses) which may occur, as was observed along lower portions of the Floodway channel.

Shear strength - effective stress relationships for Winnipeg clays have been studied in detail, with notable works by Rivard and Lu (1978), Baracos and Graham (1983), Baracos and Domaschuk (1980), Freeman and Sutherland (1973), Graham and Shields (1984) to name a few. Recent work carried out for the expansion of the Red River Floodway (KGS, ACRES, UMA 2004) included triaxial strength testing of a large number of relatively undisturbed clay samples of brown and grey clay. Samples for laboratory testing were retrieved using 100 mm (4 inch) diameter thin walled Shelby tubes. Triaxial testing consisted primarily of consolidated undrained compression testing with pore pressure measurements, and also a lesser number of consolidated drained compression tests. Figures 2-13 and 2-14 plot all measured peak and large strain effective deviator shear strengths for both the weathered brown clays and for the underlying unweathered grey clays. For comparison, the plots include both the current 2003/2004 test results, as well as test data from the 1962 PFRA investigations.

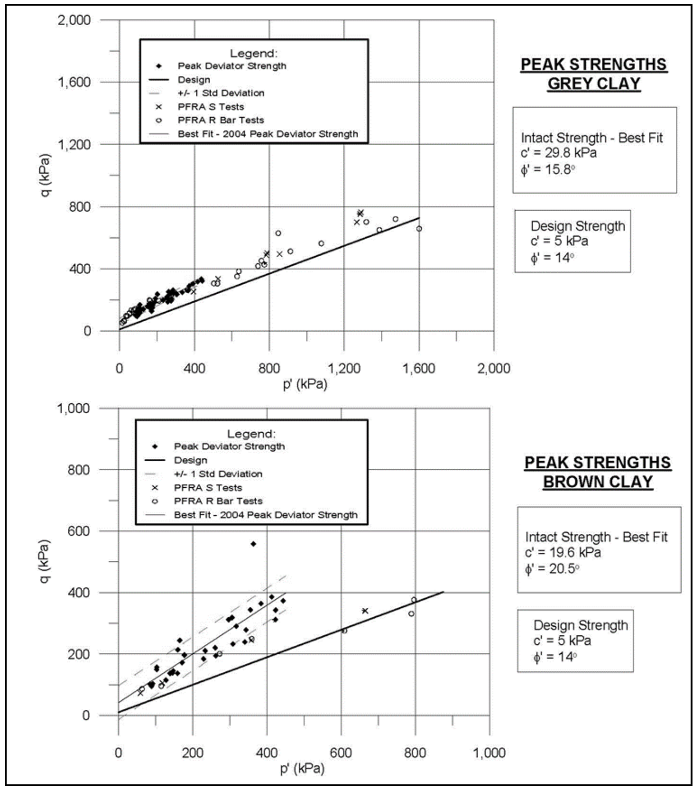

Figure 2-13 Measured peak deviator strengths (KGS, ACRES, UMA, 2004)

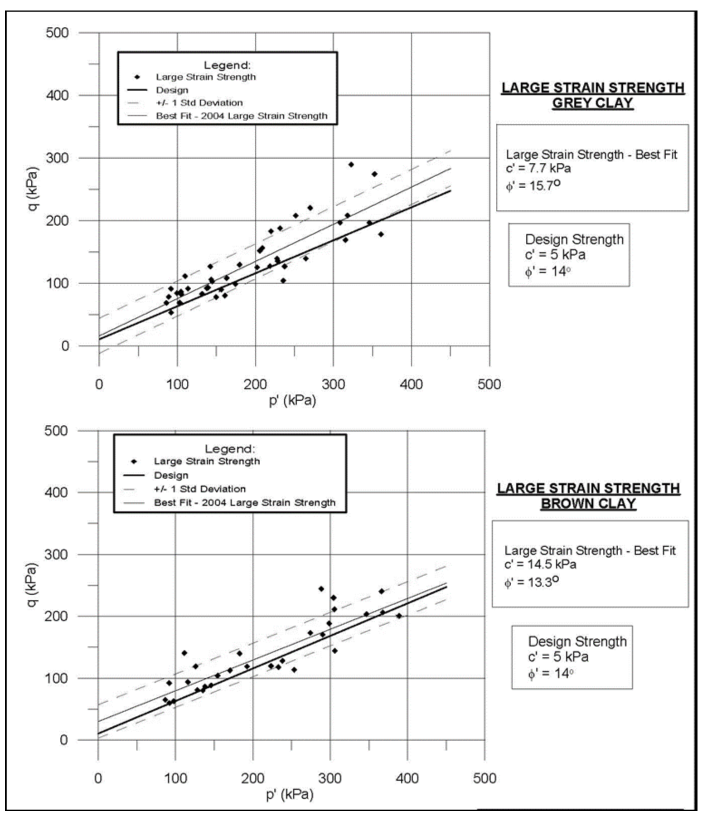

Figure 2-14 Measured large strain effective strengths (KGS, ACRES, UMA, 2004)

The design effective strength lines form a lower bound of all measured peak deviator strengths (Figure 2-13) and the approximate lower bound for virtually all the large strain effective strengths (Figure 2-14). Based on the work of Rivard and Lu (1978) and an understanding of the physical source of post-peak strengths in clays, the Author considers that in this smectitic clay, which probably has a curved post-peak strength envelope, the cohesion term c' for large strain (post-peak) used for design should probably not exceed 5 kPa without careful justification. The friction angle $\varphi'_{post\text{-}peak}$ should be adjusted appropriately to include measured test results in an appropriate stress range.

3 HYDROGEOLOGY OF THE WINNIPEG AREA

Hydrogeology (sometimes referred to as geohydrology) is the study of groundwater, specifically its movement and distribution in the soil and bedrock. Geotechnical engineers must have a good understanding of hydrogeology and the role of groundwater in the behaviour of soils and for the design and construction of geotechnical works. More often than not, groundwater is identified as a contributing factor in construction-related claims and therefore, needs very careful consideration in education and practice.

3.1 REGIONAL GROUNDWATER CONDITIONS

There are three distinct confined groundwater aquifers below the city of Winnipeg, lying between the top clay and till deposits and the Precambrian granitic basement rock (the aquifer in the Birds Hill area is unconfined). In order of importance as water supplies, the groundwater aquifers of the Winnipeg area include the ***Upper Carbonate Aquifer***, the ***Lower Carbonate Aquifer*** and the ***Sandstone Aquifer*** (Figure 3-1).

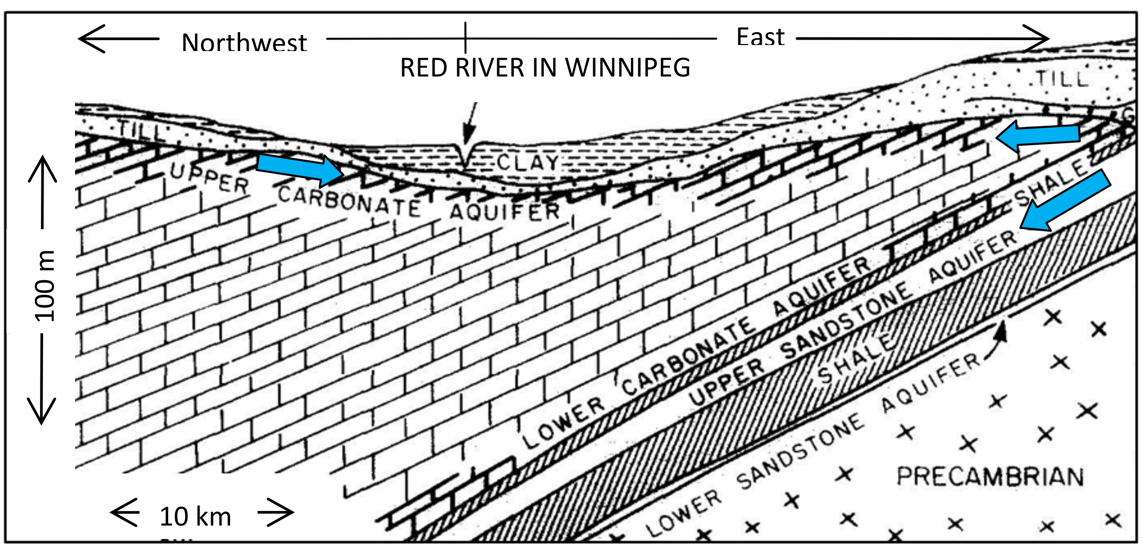

Figure 3-1 Aquifers of the Winnipeg area (modified from Render 1970)

Upper Carbonate Aquifer - The Upper Carbonate Aquifer (UCA) extends over an area more than 3400 km^2. It is confined by the overlying till and lacustrine clay deposits and by an underlying zone of low-permeability carbonate bedrock (Render 1970). It generally occurs within the top 15 to 30 metres of the bedrock, although sand and gravel, which may lie between the till and bedrock, also form part of the aquifer. The upper 7.5 metres or so of the bedrock is heavily fractured with open fissures, joints and bedding planes as seen in Figure 3-2. Large solution cavities, typical of karst topography are common and if encountered, have presented significant challenges for the construction of tunnels, shafts and deep foundations.

Figure 3-2 Deep excavation for the Red River Floodway inlet structure. *The upper carbonate bedrock exposed is known as a clint-and-grike topography with clints referring to the blocks and grikes being the fissures between the blocks (from Render 1970).*

Within the bedrock unit, this upper fractured zone has the highest permeability, with the majority of flow along fractures in the rock and thus produces the highest yields. It is believed to be a pavement karst which survived the last glaciation intact when the Laurentide ice sheet was frozen at the base, that is, the ice temperature was less than the pressure melting point (Ford 1983). This may have preserved the fragile karst landform until it was eventually covered by till and lacustrine deposits.

The transmissivity[7] (capacity to transmit water under pressure) ranges from 24.8 to 2480 m^3/m/day (Render 1970). The water quality ranges from fresh to brackish north of the Assiniboine River. South of the Assiniboine River, the water is generally brackish to saline although exceptions do exist. Salinity is a concern when groundwater pumping is necessary for excavations or where leakage may occur, for example from rockfill columns. During a period of heavy groundwater usage in the 1960s, it was reported that the boundary between the fresh and saline water was receding eastward and northward as shown on Figure 3-3 (Charron 1965).

***Lower Carbonate Aquifer* -** The Lower Carbonate Aquifer occurs along the bottom 7.5 to 15 metres of the carbonate bedrock of the Red River Formation, and immediately above the confining shale layer comprising the Winnipeg Formation. The aquifer has very low yields compared with the Upper Carbonate Aquifer and may be brackish (Ford 1982). The maximum transmissibility is estimated to be 62 m^3/m/day (Render 1970). The bedrock zone between the Upper and Lower Carbonate Aquifers yields very little water (Rutulis 1978).

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[7] Which Render (1970) calls "transmissibility" (a term now generally having been replaced with "transmissivity").

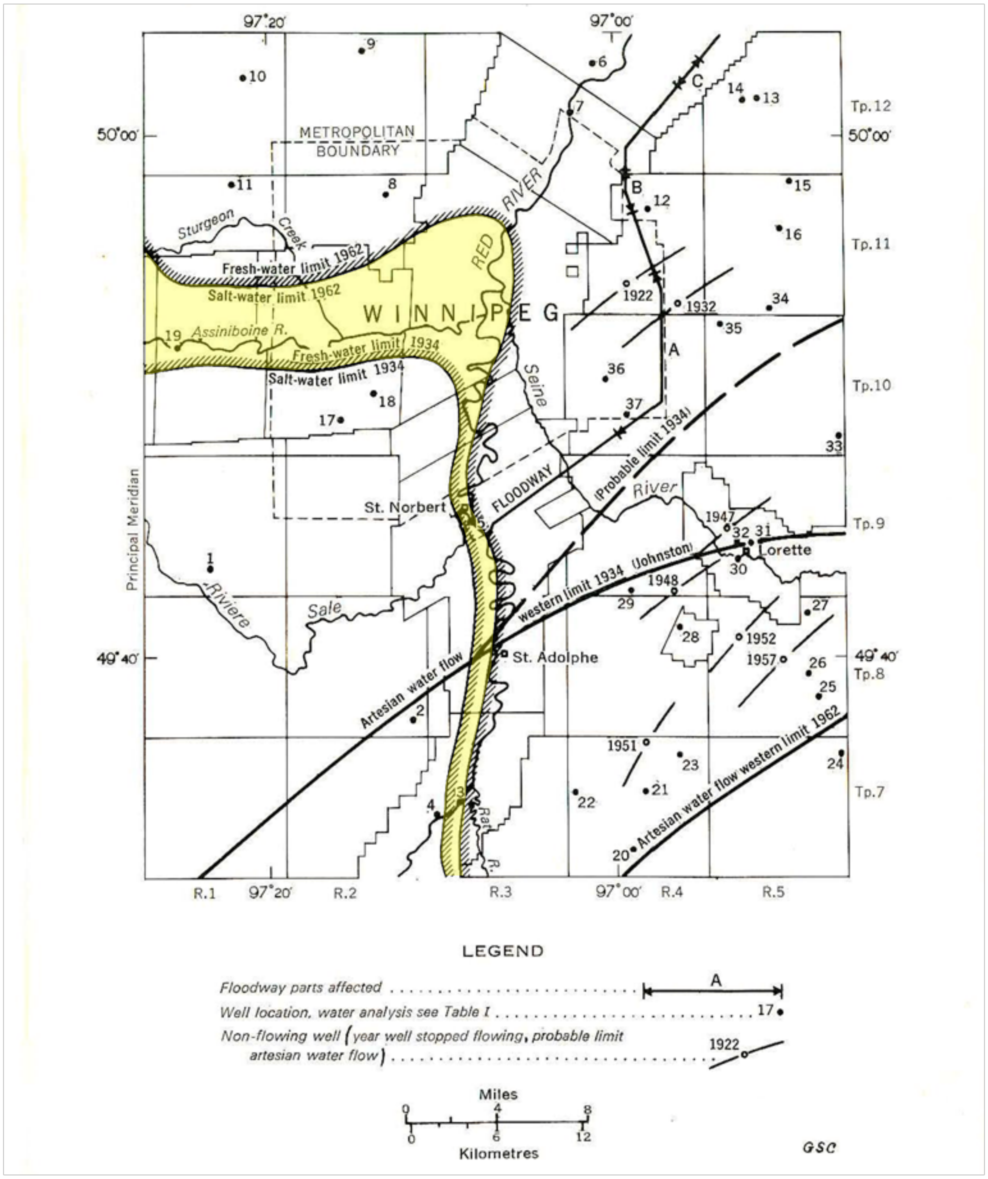

Figure 3-3 Map showing a) decrease in piezometric head, b) parts of the Red River Floodway most likely affected by groundwater, and c) boundary between freshwater and salt-water from 1934 to 1962 (modified from Charron, 1965)

Sandstone Aquifer **–** Below the limestone, the Winnipeg Formation, which is mostly shale, contains Upper and Lower Sandstone Aquifers that are separated by shale (Figure 3-1). The Upper Sandstone Aquifer consists of silicious sandstone with a transmissibility of less than 12.4 m^3/m/day. The Lower Sandstone Aquifer is coarser grained with transmissivities in the order of 124 m^3/m/day and the groundwater water is saline (Render 1970). It is underlain by the hard bedrock of the Precambrian Shield.

3.2 GROUNDWATER FLOW

Before extensive groundwater pumping began in the late 1800s, the piezometric surface[8] (also referred to as the potentiometric level) in the Upper Carbonate Aquifer was about 0.3 to 1 metre above ground surface in the north-western part of the city and 3 to 6 metres below ground surface along the Red River (Johnson 1934). Extensive pumping during the 20th century significantly depressed the piezometric surface, producing a drawdown cone over an extensive area roughly centered on downtown Winnipeg (Render 1970). A map showing the piezometric elevation, direction of groundwater flow and selected isochlors is shown on Figure 3-4 (Ford 1983). An isochlor is a line connecting points of equal chloride concentration, which in this case, illustrates the regional salinity trends.

During the period of heaviest groundwater development, the drawdown cone affected the piezometric level over an area of 3360 km^2 with a maximum drawdown of 18 to 24 metres (Render, 1970). During this period however, the piezometric level in the middle of the drawdown cone did not fluctuate significantly, indicating significant lateral recharge occurs, primarily from the following areas (Figure 3-5 and 3-6):

- The till upland area on the eastern edge of the Lake Agassiz basin,
- The Birds Hill Aquifer complex northeast of Winnipeg, and
- Thin till deposits northwest of Winnipeg.

[8] Also referred to as the potentiometric level or the level to which water will rise in a confined aquifer.

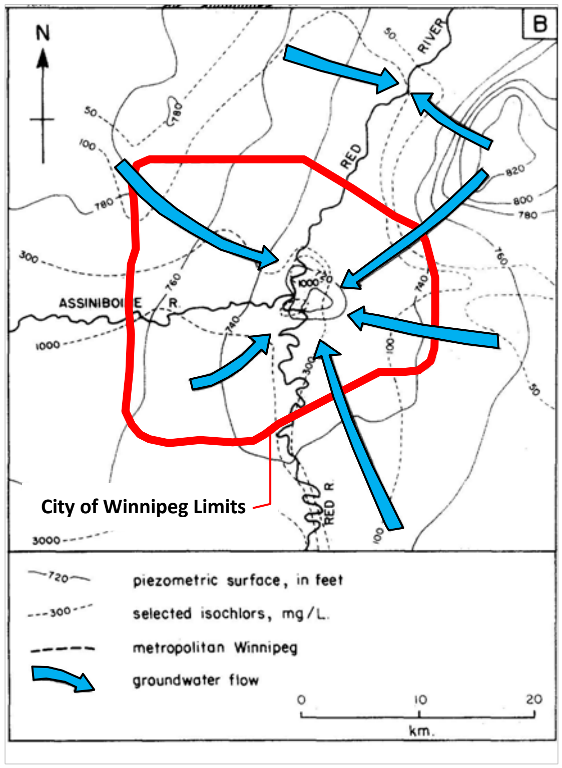

Figure 3-4 Map showing

a) piezometric surface

b) selected isochlors and

c) direction of groundwater flow in 1968

modified from Ford (1983) and based on Render (1970)

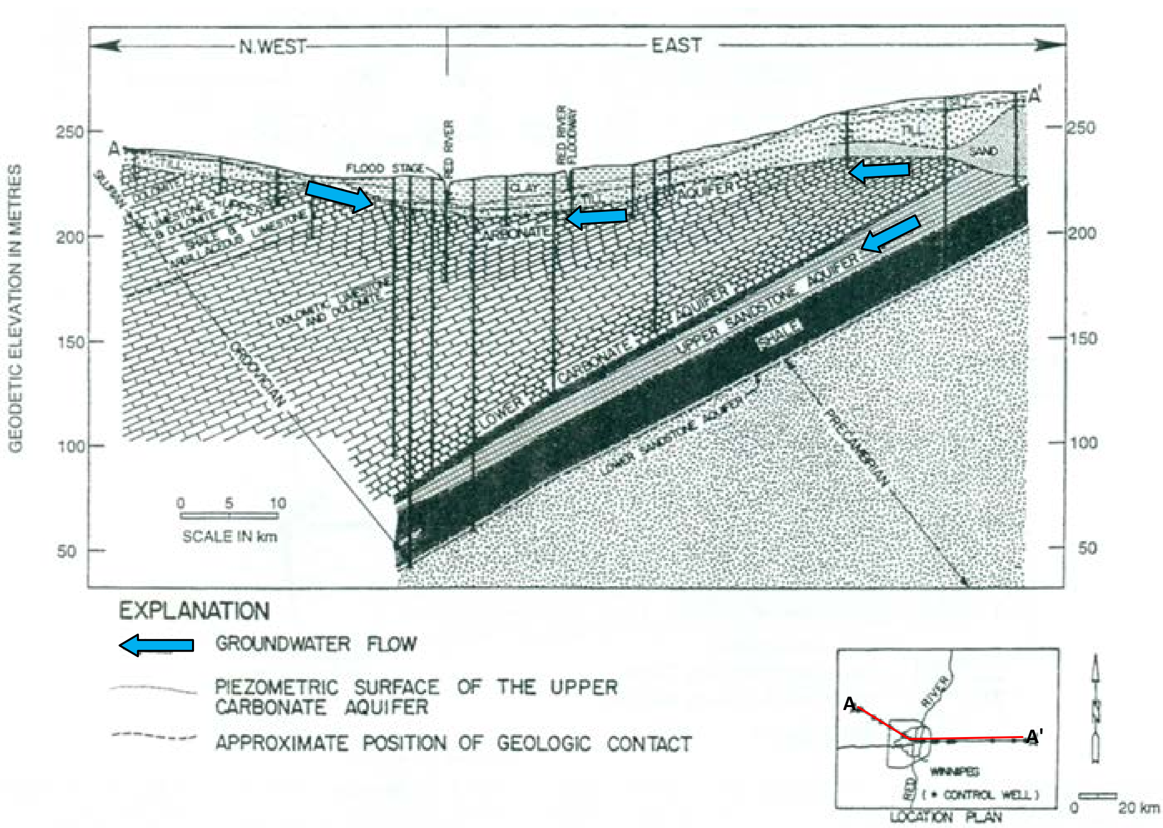

Figure 3-5 Hydrogeologic cross section A-A' through the Winnipeg area showing groundwater recharge from the east and northwest (modified from Render 1970)

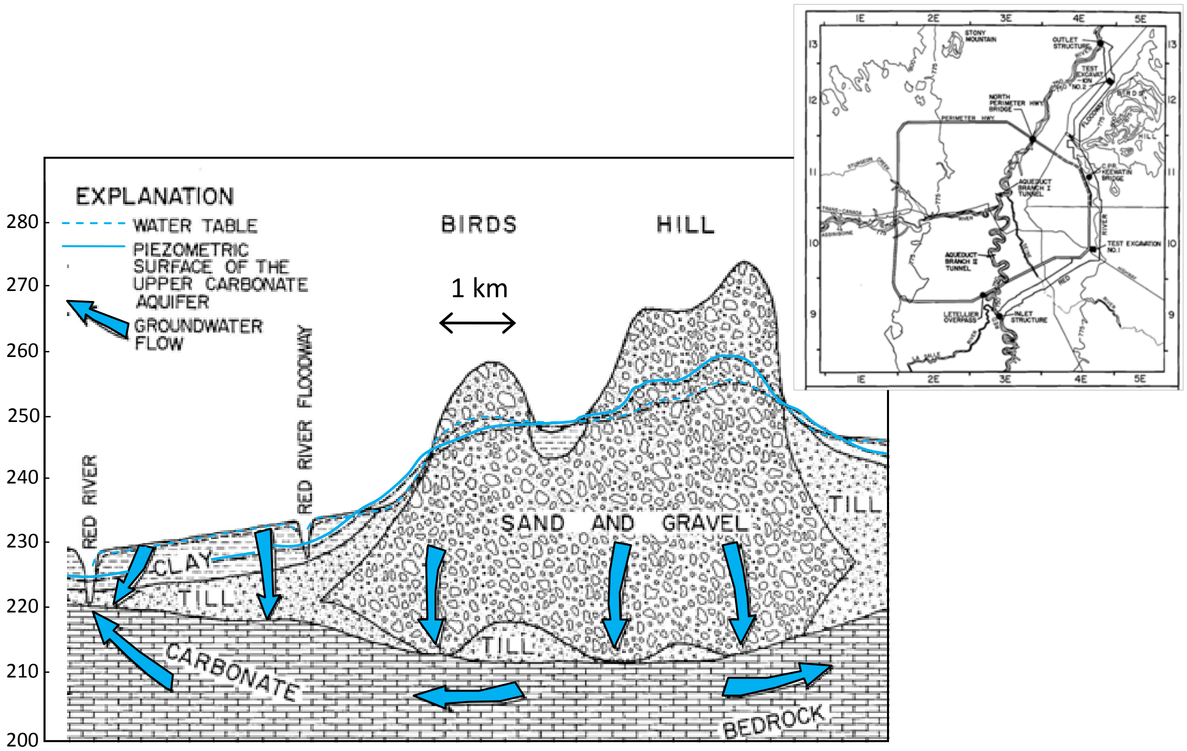

Figure 3-6 Hydrogeologic cross section (B-B'). *The section shows groundwater recharge from the Birds Hill Area to central Winnipeg (modified from Render 1970)*

3.3 GROUNDWATER USE

An understanding of historical groundwater use (consumption), primarily from the Upper Carbonate Aquifer (UCA) in the Winnipeg area is important when considering future trends in piezometric levels and groundwater quality. To put this into perspective, groundwater has been extracted from the confined UCA beneath Winnipeg for nearly 200 years. During this period, hundreds of commercial and industrial wells and thousands of domestic wells were installed (Render 1970). Historically, many consumers of groundwater pumped from the UCA without licenses or monitoring (Klassen 2010). More recently, licensed consumers have been given pumping allocations by Manitoba Water Stewardship and annual pumping amounts are now more accurately recorded.

Total estimated groundwater pumping rates and consumptive use are shown on Figure 3-7 (Render 2011).

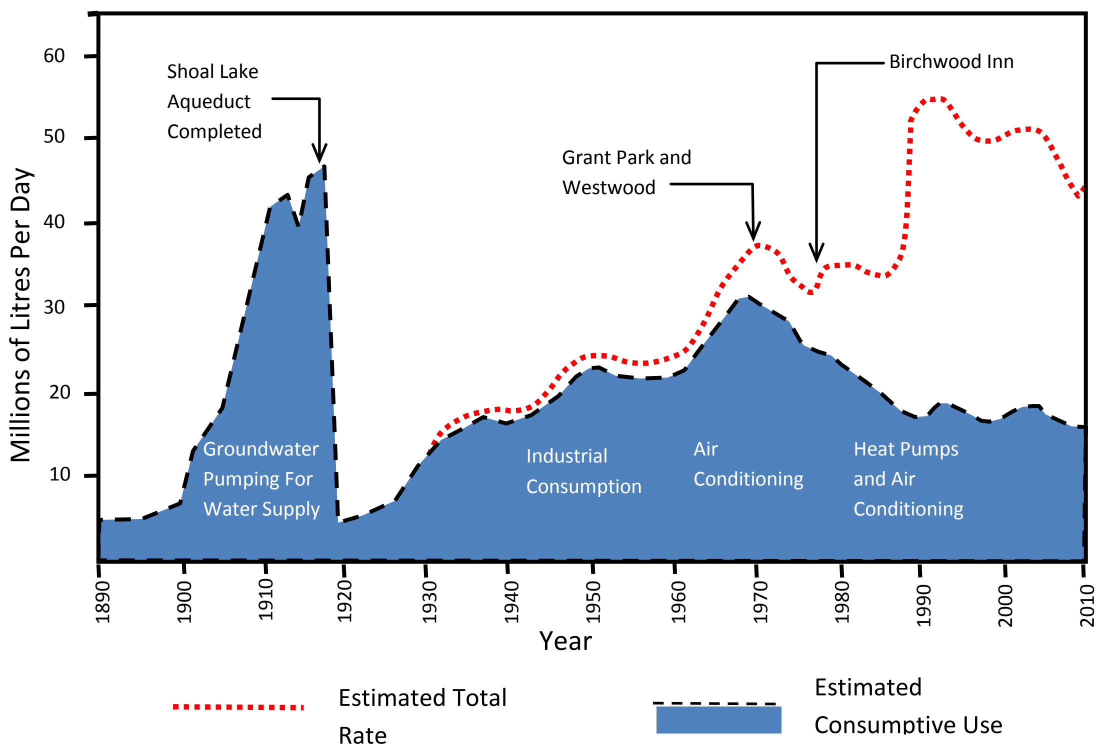

Figure 3-7 Groundwater pumping rates (modified from Render). *Several key events or changes in technology have resulted in significant decreases or increases in pumping rates. Groundwater recharge has helped offset the impact of increased pumping rates associated with heating and cooling systems.*

From 1980 until present day, pumping rates have varied from 4.5 to nearly 54.5 million litres per day. For the first 50 years of record (1880 to 1930), nearly all groundwater pumped from the aquifer was for consumption (Figure 3-7). The largest deviation in pumping rates occurred when the Shoal Lake Aqueduct was completed in 1919, in the heyday of Winnipeg's rapid growth into a major Canadian city. When it came on line, the Aqueduct, which has a design capacity of 386 million litres per day, immediately triggered a reduction in groundwater pumping by nearly 90%, from about 45 to 5 million litres per day. Following this abrupt decline, groundwater pumping rates increased due to development for industrial use (1920 to 1990) including:

- The establishment of high capacity wells for meat packing plants in the St. Boniface area of Winnipeg (Render 1970),
- Wells for air conditioning in theatres and restaurants and by cold storage plants in central Winnipeg,
- The institution of a groundwater sewage tax by the City of Winnipeg (which still exists) that encourages recharging of groundwater back into the aquifer,
- Beginning in about 1960, the installation of high capacity wells for air conditioning in for new house, apartment and hotel construction.
- In the late 1980s and early 1990s, heat pumps became more popular, triggering a further increase to about 54.5 million litres per day.

Beginning in about 1993, total pumping rates have trended downwards, mainly due to decreases in private well pumping throughout the city while at the same time, recharge rates have increased when it became a requirement for thermal effluent produced by either heating or cooling purposes to be injected back into the aquifer from which it was drawn (Render 2011). The re-injection rate is shown as difference between the red dotted line and black dashed line in Figure 3-7. One of the largest such extraction/recharge systems was installed for the Birchwood Inn in the St. James area. The effect of the increased recharge is reduced consumption and it is therefore the shaded blue area on Figure 3-7 that depicts the net extraction of groundwater from the aquifer and therefore, also explains the consequential rise in piezometric levels in the last decade or so.

3.4 IMPACT OF GROUNDWATER ON DESIGN AND CONSTRUCTION

Trends in groundwater pumping and subsequent piezometric levels in the Upper Carbonate Aquifer have significant implications on the design and construction of geotechnical works in the City of Winnipeg, including deep foundation installation and excavations (Render 1970). Of particular concern is the gradual but steady rise in piezometric levels in the drawdown cone associated with a sizable reduction in groundwater consumption. Rising groundwater levels may cause problems such as base heave or excessive seepage during construction and they may also impact projects completed many years earlier. Another significant concern is the increased pore water pressures in the till leading to a reduction in shear strengths along the clay-till interface, a condition which can contribute to deep seated slope instabilities. This chapter discusses the nature of the problem while subsequent chapters will deal with the consequences and solutions.

Rising piezometric levels at any location in the City of Winnipeg depend on several factors, most notably the location with respect to the drawdown cone, and the discharge and recharge in the immediate area. Problems that may be encountered are largely dependent on hydraulic conductivity and transmissivity of the aquifer at the site. Intuitively, the magnitude of recovery will be greatest at the middle of the cone and the least at the outer fringes. For example, water levels have risen by about 3 metres in the central part of the City over the last 30 years, primarily due to decreases in consumptive use. A significant amount of data on groundwater levels is available from measurements in provincial observation wells throughout the City. The frequency of monitoring ranges from daily in some wells to only a few times each year in others (Klassen 2010). Contour maps of piezometric levels are also created on a regular basis, beginning in 1965. Figure 3-8 shows an example of a piezometric elevation map from 1970. Differences in piezometric elevations that have occurred from 1970 to 2009 are illustrated in Figure 3-9.

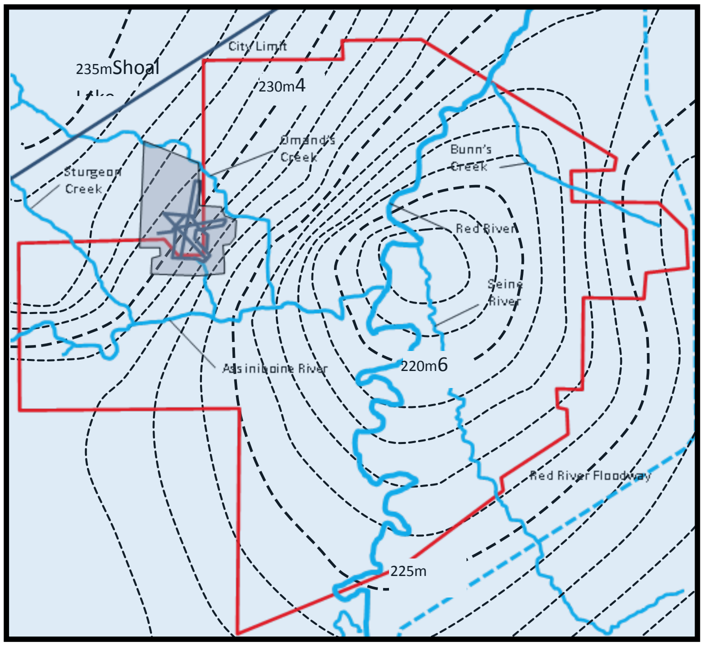

Figure 3-8 Potentiometric surface in the Winnipeg area in 1970

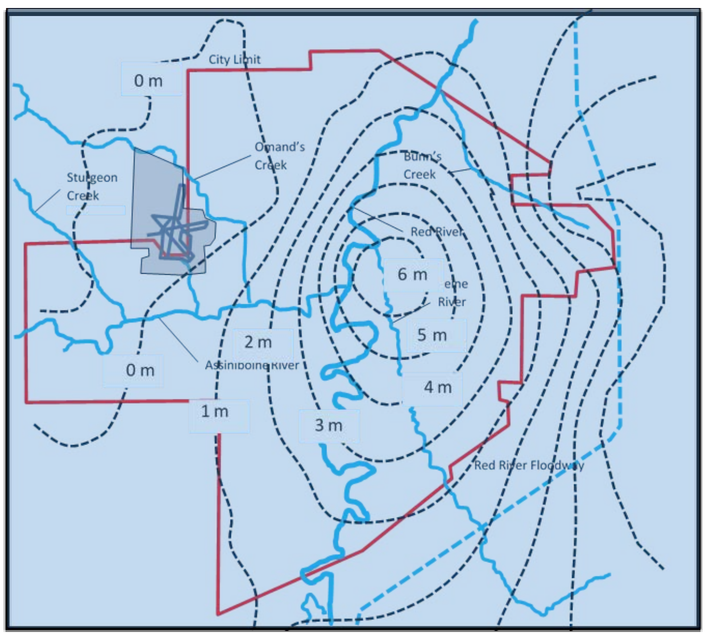

Figure 3-9 Change in Potentiometric Surface in the Winnipeg area from 1970 to 2009

Klassen (2010) carried out an investigation of the influence of groundwater levels on riverbanks in the Winnipeg area. As part of this work, available data on groundwater levels were summarized and plotted to create contour maps of the piezometric surface based on groundwater levels in the provincial observation wells. The greater Winnipeg area was divided into three zones centered on the downtown area where the greatest drawdown has occurred historically (Figure 3-10). The average annual groundwater levels were then plotted for each zone over time as shown on Figure 3-11 which shows the trends in each with respect to recovery of groundwater levels. The most significant change (about 3 m) occurred in Zone 1 at the middle of the drawdown cone with little to no change in groundwater elevations in Zone 3.

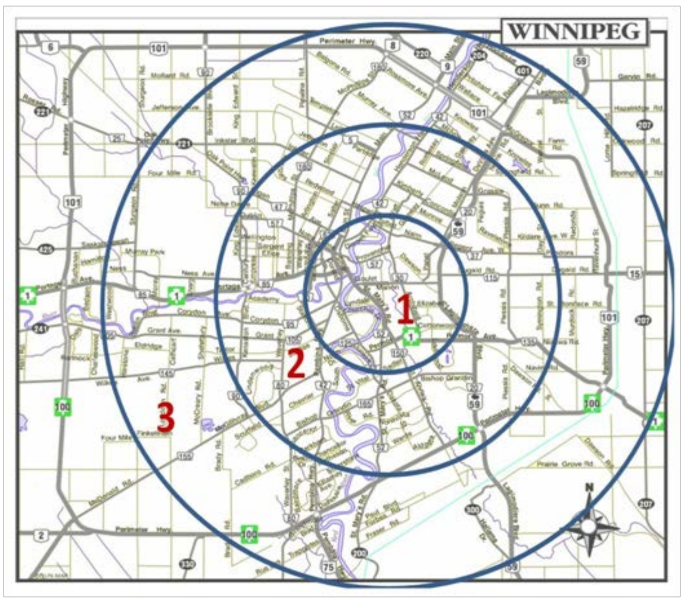

Figure 3-10 Zones 1, 2, and 3 (Klassen 2010). *Zone 1 is centred on downtown Winnipeg where the effects of drawdown are most significant. Zone 3 extends to or just beyond the city limits.*

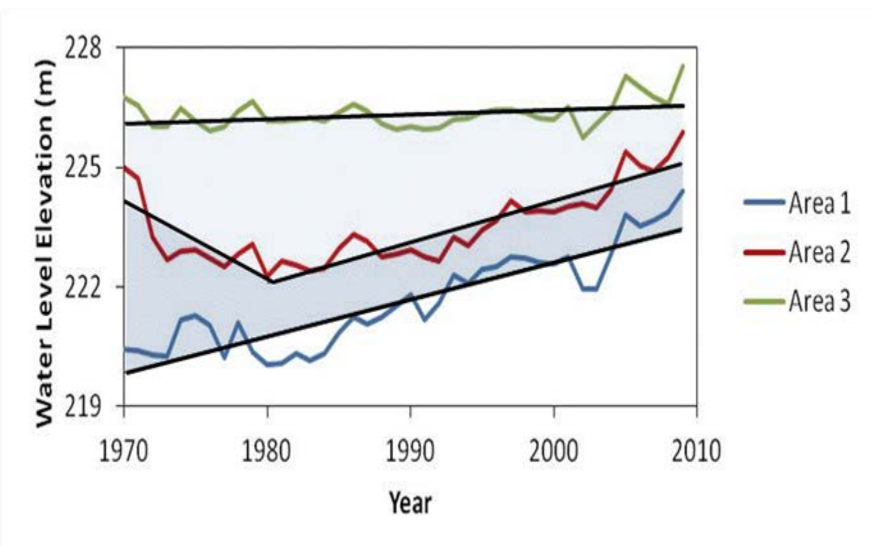

Figure 3-11 Average annual groundwater levels in Zones 1 to 3 (Klassen 2010)

The change in the shape of the drawdown cone is best illustrated in a west to east profile through central Winnipeg as shown on Figure 3-12 just north of the Assiniboine River (from Klassen 2010). The region between the lowest and highest piezometric surfaces has been shaded to help picture the pool of water rising within the bedrock aquifer. Also shown is the bottom of the Red River channel relative to levels recorded in 2008.

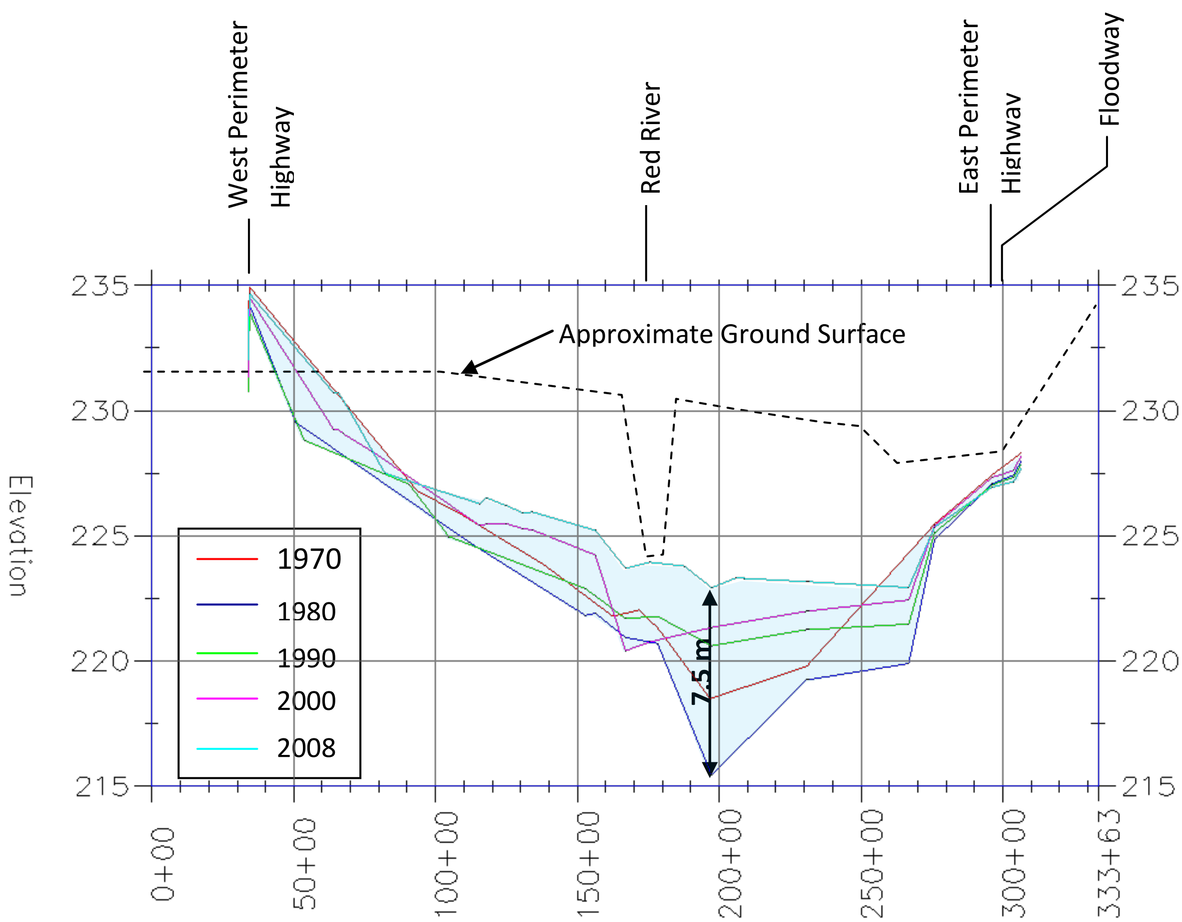

Figure 3-12 West to east profile of piezometric levels beneath Winnipeg (modified from Klassen 2010). *The implications are obvious for deep excavations (for example).*

Groundwater in the till is often encountered in deep foundations such as caissons and in excavations where the depth of cover is limited as it is in northwest Winnipeg. The groundwater is typically found in the upper ablation till (if present) or less frequently within permeable sand and gravel layers within the more dense basal till. Depending on the location within the city, piezometric levels in the till may be influenced by the piezometric levels in the overlying clay, underlying bedrock, river levels, and levels within major drainage channels such as the Red River Floodway.

For example, during expansion of Winnipeg's Water Treatment Plant in 2005, piezometric levels in the till responded almost immediately to rising levels in the Floodway channel when it was operating. Piezometric elevations in monitoring wells installed in the work area rose to within about 0.5 metres of the water level in the channel, located about 250 m west. The higher elevations required a series of pumping wells to lower the groundwater level during construction. The till at this location is about 17 m below ground surface and about 4 or 5 m below the bottom of the Floodway channel. A similar response has been seen at the Transcona Grain Elevator, located more than 3 km west of the channel. Here, seepage into the sump pit in the basement of the elevator increases in response to increased levels in the Floodway channel (personal communication, Bill Parrish). It is likely that this occurrence is due to a hydraulic connection that was created between the clay and till during righting and underpinning of the structure in 1914. The hydraulic connection between the Floodway channel and the till likely occurs several kilometres downstream where the channel intercepts glaciofluvial deposits of the Birds Hill Aquifer.

Groundwater levels in the lacustrine deposits across Winnipeg are typically within about 2 or 3 metres below ground surface although exceptions do exist. Most excavations into the clay, including shafts for deep foundations are carried out with no physical evidence that the excavation has advanced below the groundwater table. This is due to the very low permeability of the clay which typically ranges from an average of 2.1 $x10^{-9}$ m/s in the upper brown clay (Day 1977) to an average of 1.0 x 10^{-12} m/s in the lower grey clay (Baracos 1960, Mishtak 1964). An exception is the silt layer in the Upper Complex Zone that may contain a perched groundwater level. Seepage and sloughing conditions are often encountered in excavations and shafts in the silt layer when it is in a wet condition.

Slope stability is highly sensitive to the position of the groundwater table or more specifically, the piezometric surface within the clay and the gradient between the clay and till. The relationship between the piezometric levels in the till and lacustrine clay can be complex and has been shown to be dependent on river levels. For example, there may be a downward gradient between the clay and till during normal to high river levels and an upward gradient between these two units during low river levels (Thiessen 2010, KGS 1994). This condition can result in a reduced stabilizing pressure at the toe of the slope (Graham 1986, Tutkaluk et al 2002). The application of this observation relative to slope stability analysis is discussed in more detail in Chapter 7.

4 SUBSURFACE INVESTIGATIONS

The properties of soil, rock and groundwater play an important role in the design, construction and ultimate safety of nearly all civil engineering structures. The materials either provide support for structural loads or exert pressures on structural components such as walls, dams and tunnels. Many civil engineering failures can be tied back to the nature or action of geotechnical conditions that were incorrectly or inadequately defined or poorly interpreted. What separates successful projects from those where failures occur is often a careful and accurate subsurface investigation. Conversely, a poor subsurface investigation can easily lead to failure, or at the very least, to an uneconomical or over-designed structure or construction-related claim.

A subsurface investigation needs to provide a reasonably accurate representation of soil, rock and groundwater conditions such that the geotechnical engineer can extract the necessary information and produce a safe and economical design. Often, this information is provided to an owner who may retain an independent structural engineer to carry out a design based on parameters provided in the geotechnical report. A contractor will then often use information in the report or on test hole logs sheets for bid estimates and ultimately for project planning. Within this process, the information from the site investigation will often be utilized or perhaps interpreted by many parties, some more familiar with the technical information provided than others.

Depending on the project requirements, a subsurface investigation may be either preliminary or detailed. In some cases, a conceptual level investigation is required; a subsurface investigation for this level of study would be very limited or non-existent, relying rather on available geotechnical and geological maps and other related sources. In almost any case, a review of existing information can provide valuable information to assist in planning an investigation or providing supplemental information for design. Many subsurface investigations are carried out in one stage due to budget and schedule constraints or where they themselves are sufficient for the project requirements, for example for borrow investigations, relatively simple foundation designs, or slope stability problems.

4.1 REVIEW OF EXISTING INFORMATION

There are several sources of information available for use in planning subsurface investigations in the Winnipeg region. Most notably are the maps prepared by Kjartanson et al (1983) which summarize the results of an extensive compilation of geological and geotechnical data from the city of Winnipeg area in a series of Map Sheets that include depth to till, depth to power auger refusal, depth to bedrock and potentiometric levels in the Upper Carbonate Aquifer. The maps are more accurate in areas of historical development such as the downtown area and less so in areas where little to no development had taken place at the time of publication. These volumes are out of print but should be sought out by practicing geotechnical engineers as a valuable resource.

Hundreds of geotechnical reports have been written describing bedrock, soil and groundwater conditions in the Winnipeg region. While these reports may not be readily available, it is worth searching for them through property owners past and present. In many cases, a link between a project site and local consultant can be found and if the original client can be contacted, the geotechnical report may be made available. This is not to say that the information contained in a previous report should be used for design (in fact it is generally expressly prohibited), but that it may provide valuable information relative to subsurface conditions and therefore guidance for the planning of a current subsurface investigation.

Aerial photos are available from a number of sources and these may be useful in determining historical land use, drainage features, rates of erosion and previous slope instabilities along riverbanks for example. With a legal property description (Section, Township and Range are the best), the air photo library operated by Manitoba Conservation (1007 Century Street) provides aerial photography of Winnipeg and Manitoba from as far back as 1928 to present day. For sites located along the waterways, the Waterways Section of the City of Winnipeg (part of Planning, Property and Development Department) has numerous years of aerial photography, generally taken in the fall during low river levels. With permission of the Waterways Engineer, the photos may be available for viewing, or alternatively, digital aerial photography from 2008 and 2013 can be purchased.

4.2 PRELIMINARY SUBSURFACE INVESTIGATION

A preliminary investigation may be requested in order to evaluate possible locations for a structure or determine potential foundation alternatives for a structure. It is generally limited to a very broad geological reconnaissance with limited subsurface investigations and sampling for obtaining general observations of soil types, depth to bedrock and groundwater conditions. Clearly, obtaining even some very preliminary information can greatly reduce the uncertainty about the soil and groundwater conditions and can avoid unexpected project expenses.

The preliminary subsurface investigation is typically carried out early in the project timeline, often before structural elements have been finalized, or specific locations for foundation elements, earth fills, retaining structures, etc. have been determined. Typically only a limited number of test holes are drilled and soil sampling/testing focus on obtaining a general indication of soil stratigraphy and groundwater conditions relative to important features of the project. The work should be carried far enough forward to assess viable foundation options (for example), identify important design issues, and allow for appropriate planning for detailed investigations.

4.3 DETAILED SUBSURFACE INVESTIGATION

The primary purpose of a detailed subsurface investigation is to refine the site characterization made from the preliminary subsurface investigation and also to reduce the risk of encountering unexpected conditions during construction. The investigation is typically carried out once a determination of structural elements has been made. It obtains specific subsurface information for ponds, embankments, retaining structures, and at the final location of foundation units. The detailed investigation typically includes physical sampling of soil and rock through drilling or test pits, a laboratory testing program to determine the engineering properties of samples, and often the installation and monitoring of geotechnical instrumentation.

4.3.1 Location of Boreholes

The term "borehole" is often used interchangeably with "test hole", or in older reports, the term "borings" may be used. Boreholes are drilled into the soil or rock using a variety of methods to facilitate visual classification of the subsurface strata and obtain disturbed and undisturbed samples. Test pits may replace boreholes[9] if the depth of investigation is relatively shallow (less than say 6 metres) but note the importance of installing adequate support to protect those inspecting the sidewalls of the pit.

Once a preliminary investigation has been carried out or a general understanding of the project requirements has been determined, the next step is to determine the location and spacing of boreholes. In this regard, the most important consideration is for the subsurface investigation to be sufficient to reasonably characterize the type and extent of soil or rock masses, determine rock quality, identify important irregularities, and assess short term groundwater conditions.

The layout of boreholes should depend on the size of the structure and expected variability of subsurface conditions. Where significant variability is anticipated, the borehole spacing should be reduced. If the location of foundation units or excavations on site has not been finalized, a minimum of three boreholes in an approximately triangular pattern and ideally, at a maximum horizontal spacing in the order of 15 metres is suggested. In most cases, this spacing should provide adequate characterization of subsurface conditions even for erratic conditions. A triangular array may also be sufficient to determine the horizontal groundwater flow direction if piezometers are installed in each hole.

If the location of structures is known, the boreholes should be targeted around major foundation units and preferably in proximity to the corners and perhaps one in the center of the structure. Wherever possible, it is advisable to locate boreholes outside of the

[9] The term "boreholes" will be used throughout this thesis.

perimeter where foundation units will be located as failure to properly seal off boreholes may result in groundwater seepage during construction. If this is not possible, care should be taken to backfill the borehole with impermeable soil, solid bentonite seals (pellets or chunks), or bentonite or bentonite-cement grouts. Large boreholes, for example those done using piling rigs, are typically backfilled with clay cuttings (rammed in place) or with a sand-bentonite-cement ready-mix backfill. The advantages and variations of backfill mixes are described by Mikkelsen (2002).

The Canadian Foundation Engineering Manual (CFEM) also provides guidelines for borehole spacing based on the size of a structure. For buildings with footprints ranging from 250 to 1000 m^2 it recommends that between four and five boreholes be drilled. For buildings less than 250 m^2, three boreholes are suggested. If in doubt, drill an additional borehole.

4.3.2 Depth of Exploration

Boreholes or test pits should be deep enough that the entire zone of rock, or rock affected by loading changes, is adequately explored (CFEM 2006). The depth of exploration depends to some extent on the size of the structure, and must give due consideration to the possibility of foundation failure, excessive settlement, seepage and earth pressure. However, to a larger degree, it also depends on the type of the foundation soil (Hvorslev, 1965). The depth of exploration needs to extend to a stratum of adequate bearing capacity and should almost always pass entirely through layers of unsuitable soil such as silt, organics, and fill. Depending on the depth of the foundation and size of the structure, consideration should be given to extending boreholes through the entire compressible clay layer, in particular strata below the upper over consolidated brown clay.

Foundation Structures

A common rule-of-thumb is to extend boreholes to a depth where the net increase in stress from a structure, embankment, etc., is less than 10% of the applied load or less than 5% of the effective stress in the soil at that depth, whichever is less. This relatively easy guideline is illustrated on Figure 4-1. In the case of compressible lacustrine clays in the Winnipeg region, it is generally preferred to extend the boreholes deeper than the 10% and 5% rule, often to the clay-till contact[10].

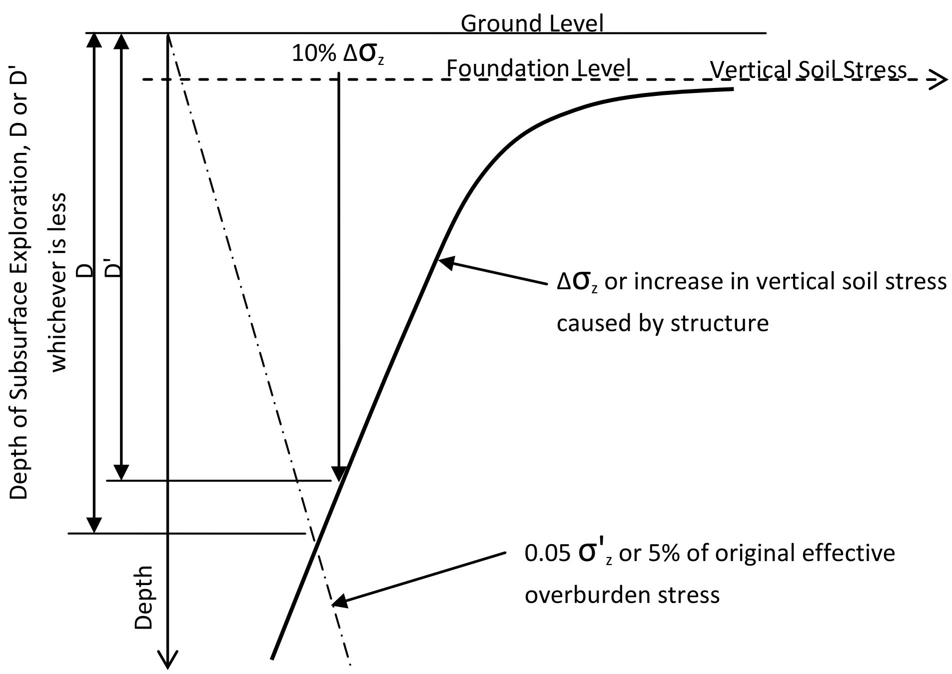

Figure 4-1 Depth of exploration for structural loads

In addition to the rule-of-thumb in Figure 4-1, it should also be noted that the minimum depth of exploration below a foundation element should be 6 metres unless a hard stratum such as till or bedrock is encountered first. If till is encountered, the borehole should be advanced to power auger refusal, anticipating that driven end bearing piles or cast-in-

[10] An important consideration in this rule-of-thumb is the tendency for shear strengths to decrease with increasing depth in Winnipeg.

place caissons may be under consideration. If bedrock is suspected, a minimum of 3 metres of core should be taken to preclude (but not necessarily eliminate) the presence of a boulder or weaker layer below the bedrock surface. For rock-socketted caissons, the depth of core should be at least 3 metres below the anticipated base of the socket. If there is any doubt, drill deeper.

Retaining Walls

Often the governing concern is bearing capacity failure and in this regard, the depth of exploration should be greater than the depth to which a failure surface may occur. For most cases where the wall is supported on lacustrine clay, a depth of exploration of D=2H is recommended where H is the wall height. If the wall is high enough where significant settlements are expected, the depth should be increased accordingly (perhaps following the rule-of-thumb for structure foundations).

Fills

Fills such as approach embankments may fail in bearing capacity was well as excessive consolidation settlement. Maximum shearing stresses generally occur at a depth of D = 1.25L or D = 0.5L depending on the geometry (Hvorslev 1949). The two cases are shown on Figure 4-2 where L is average horizontal length of the sideslope. This does not preclude the possibility that failure surfaces can extend deeper, but these guidelines should at least be sufficient for preliminary planning. For embankments greater than 4 or 5 metres high, vertical and shear deformations may govern and the entire thickness of the compressible clay layer should be explored.

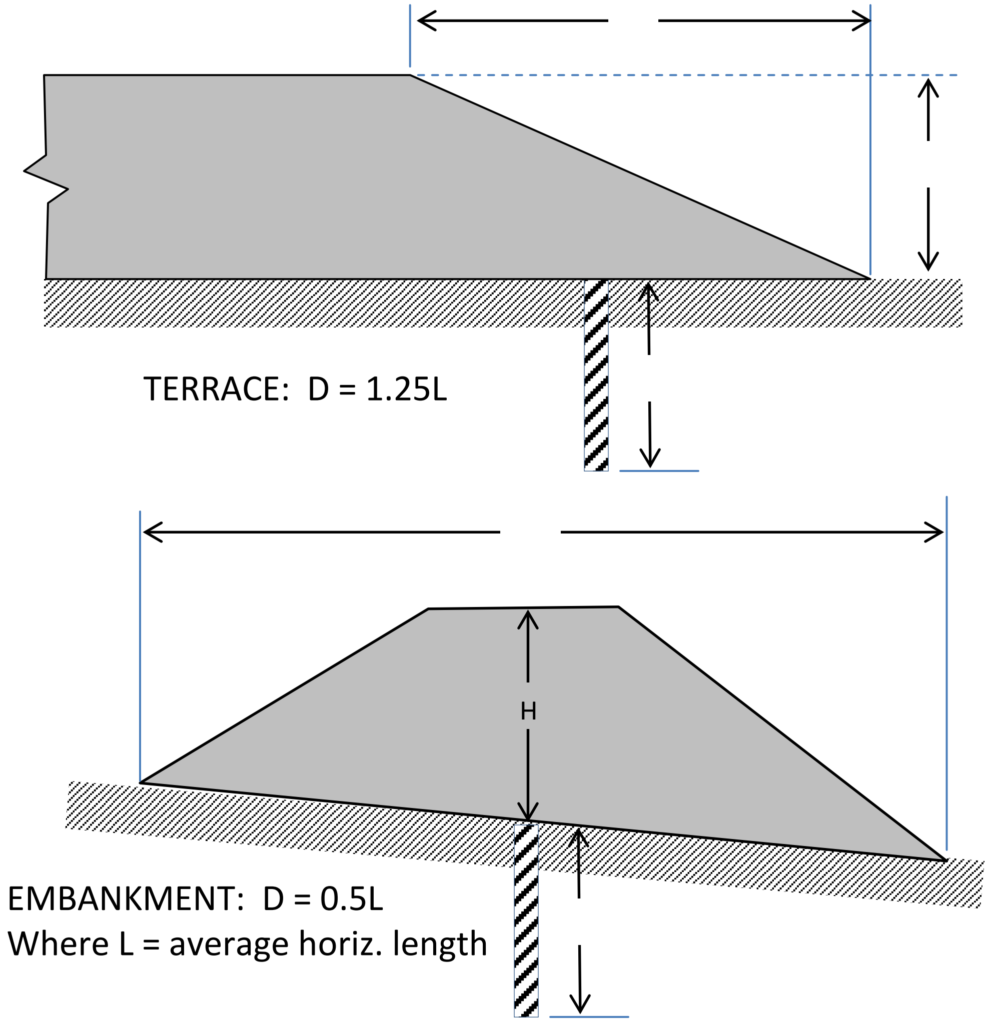

Figure 4-2 Depth of exploration for terraces and embankment fills (based on Hvorslev 1949)

Cuts

The governing concern for deep excavations is a sideslope failure where the depth of the failure surface depends on a number of geometric factors, the shear strength of the soil, and pore water pressures near the base of the excavation. In this regard, it is recommended that preliminary stability analysis be carried out to determine the maximum depth of potential failure surfaces and failing that, plan on extending the depth of exploration to twice the excavation depth unless it can be demonstrated through modelling that the base width of the excavation is sufficiently small to limit the depth of a slope failure.

Roads, Parking Lots, and Airstrips

In general, a depth of exploration of 1.5 to 3 m is sufficient, with the maximum depth often determined by the thickness of the near-surface silt layer. It is often desirable to drill to about 0.5 m below the bottom of a silt layer to provide an adequate characterization of this material within the Upper Complex Zone. The borehole spacing is often set at 50 m with infilling as required to better delineate areas of problematic soil, for example shallow silt layers.

4.4 DRILLING METHODS AND SOIL SAMPLING

The type and frequency of soil sampling depends on the project requirements and drilling method used. Continuous-flight solid-stem auger rigs and piling rigs are two of the more common drilling methods in Winnipeg. The continuous-flight auger allows disturbed samples to be brought to surface at intervals of about 1.5 m for classification and sampling providing the augers can be pulled out without turning (Figure 4-3). This method is successfully used in most lacustrine clay soils but may not work well in alluvial soils or in wet silt layers. It may be possible to keep a hole produced by a piling rig open in such conditions by back-spinning the auger to smear the borehole wall with clay or by using steel casing to seal off near-surface wet layers prone to sloughing.

Figure 4-3 Track mounted drill rig with 125 mm diameter solid stem augers. *Depending on the height of the mast, up to 3 metres of auger can be pulled straight out of the borehole for sampling purposes.*

With smaller diameter augers, it is difficult to observe down-hole conditions such as seepage and sloughing from wet layers. In a larger diameter borehole drilled with a piling rig, a light can be lowered or a mirror used to reflect light down the hole to make observations and measure the depth to different soil layers or seepage zones. Compared with solid-stem augers, piling rigs can usually advance farther through cobbles and small boulders that may be encountered in the till, or penetrate farther into dense basal till. This makes the piling rig more suitable for assessing the refusal depth of driven end bearing piles. Figure 4-4 shows a piling rig used for geotechnical investigations.

Disturbed (grab) samples can be taken directly off the auger or thin-wall ‘Shelby’ tubes are pushed to recover relatively undisturbed samples. Care must be taken (in particular with a piling rig) to avoid over-pushing Shelby tubes. Standard penetration testing (SPT) can be carried out in the clay, although this is not generally the preference of local practitioners. Results of SPTs are difficult to interpret in clays.

Figure 4-4 Piling Rig Borehole. *The auger has been removed and a Shelby tube is being pushed. The photo inset illustrates the ability to visually assess soil and groundwater conditions in the open hole. Surficial gravel fill is visible near surface and water can be seen reflecting light at the bottom of the hole.*

If sloughing conditions are encountered or anticipated, it may be necessary to utilize hollow stem continuous flight augers (Figure 4-5). This drill method is suitable for almost all soil types but is of particular value in saturated silts and sands that would not otherwise stand open. Auger flight sampling is not possible; sampling is carried out by advancing either a split spoon or Shelby tube fixed to the end of the drill rod. Care must be taken when pulling the plug at the end of the auger string for sampling as saturated non-cohesive soils can blow up into the auger greatly disturbing the sample and making it very difficult to reinstall the plug. It is good practice to keep the head of water inside the auger at or higher than the groundwater level in the strata being sampled. To maintain this level, water must be continuously added as the drill rod and plug are pulled.

Figure 4-5 Track mounted drill rig equipped with hollow stem augers.

Geotechnical instrumentation can be more easily installed in boreholes drilled using solid or hollow stem augers, compared with the 300 to 450 mm diameter holes that are commonly drilled using a piling rig auger. With the smaller augers, once the instrumentation is in place, only a small volume of backfill or grout is required for the installation.

Once augers can no longer be advanced into dense till or bedrock, rotary methods must be used if deeper exploration is required, for example, for rock-socketted caissons. Care must be taken when coring through the till as drill fluid may disturb the finer grained portion of the till matrix and limit the ability to recover good quality samples. Standard Penetration Test blow counts can be measured in the till, although the presence of gravel and cobbles often hinders the test. Coring through the limestone bedrock is relatively easy, although recovery can be difficult in heavily fractured zones. Care should be taken to properly seal off boreholes advanced into the till and bedrock to prevent possible hydraulic connections with near-surface works, for example excavations, or to avoid contamination of the Upper Carbonate Aquifer.

4.5 A WORD ON SHELBY TUBE SAMPLING

A thin walled Shelby tube is commonly used to obtain samples of cohesive materials. The tube is attached to the end of a drill rod and pushed into the soil without any rotation or chopping action and without removing any of the soil that is displaced by the steel cylinder as it advances. In this fashion, the soil is pushed out of the way resulting in the development of significant stress changes and plastic deformation of the soil (Hvorslev, 1949). Perhaps by virtue of describing the sample as "relatively undisturbed" Geotechnical Engineers may inadvertently disregard these effects and not give due consideration of possible ways to minimize them.

Work by Casagrande between 1925 and 1936 demonstrated the effects of sample disturbance. Hvorslev (1949) provides an excellent analysis of the forces acting on the soil while a drive sampler (Shelby tube) is being pushed as shown on Figure 4-6. The water or air pressure at the top of the sample (U_t) and the friction along the inside of the tube (F_i) cause an increase in pressure (P_e) over the surface area at the end of the tube (A_e). The pressure (P_p) on the surrounding annular area (A_p) becomes very large due to edge resistance (Q_p) and because the soil must be displaced as the sampler advances through the soil. The pressure on the outside of the tube (P_o) is a function of the overburden pressure and friction along the outside of the tube.

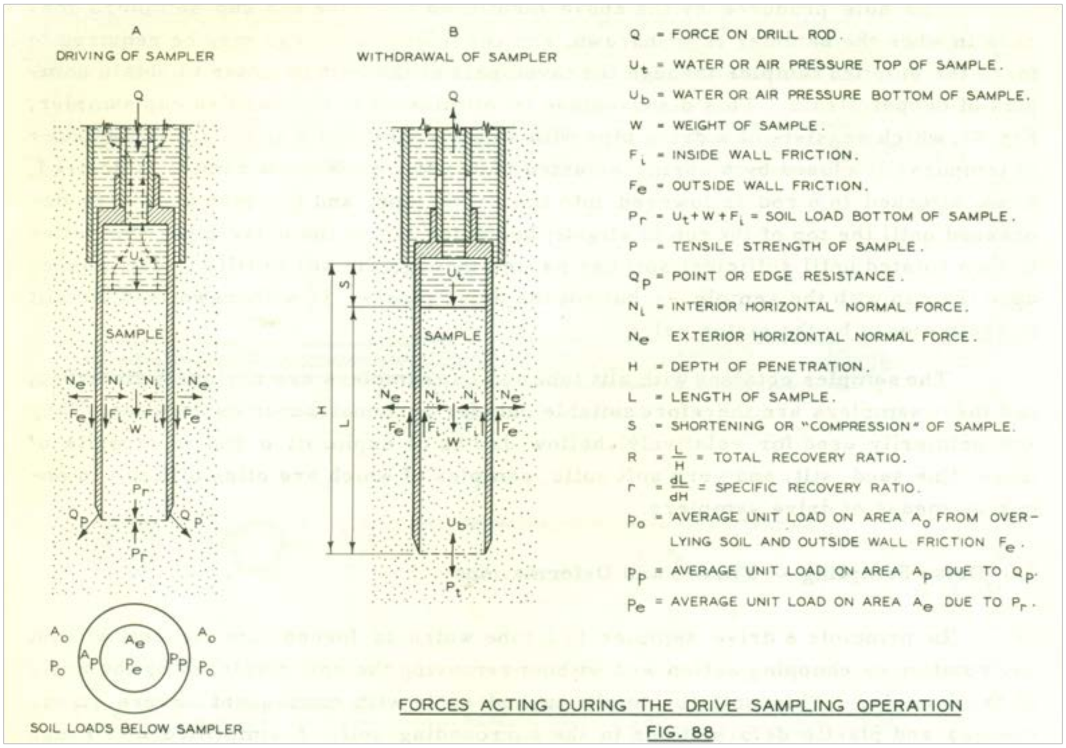

Figure 4-6 Forces acting on Shelby tube during pushing (Hvorslev 1949)

With continued advancement, P_e can increase and exceed the bearing capacity of the soil. This causes shear failure of the soil as the soil is deflected downward. These shear planes can be captured in the sampling tube and may help explain the presence of what appear to be slickensides in samples collected at depth in flat prairie regions (Figure 4-7). Forces during withdrawal of the sampler can also be significant when the wall friction is reversed. To retain the sample, F_i + U_b must be greater than U_t + W + P_r. Since a reduction in the upward pressure U_b is often the cause of sample loss, a check valve in the Shelby tube adaptor at the end of the drill rod can be used to reduce the water pressure at the top of the sample and aid in sample recovery.

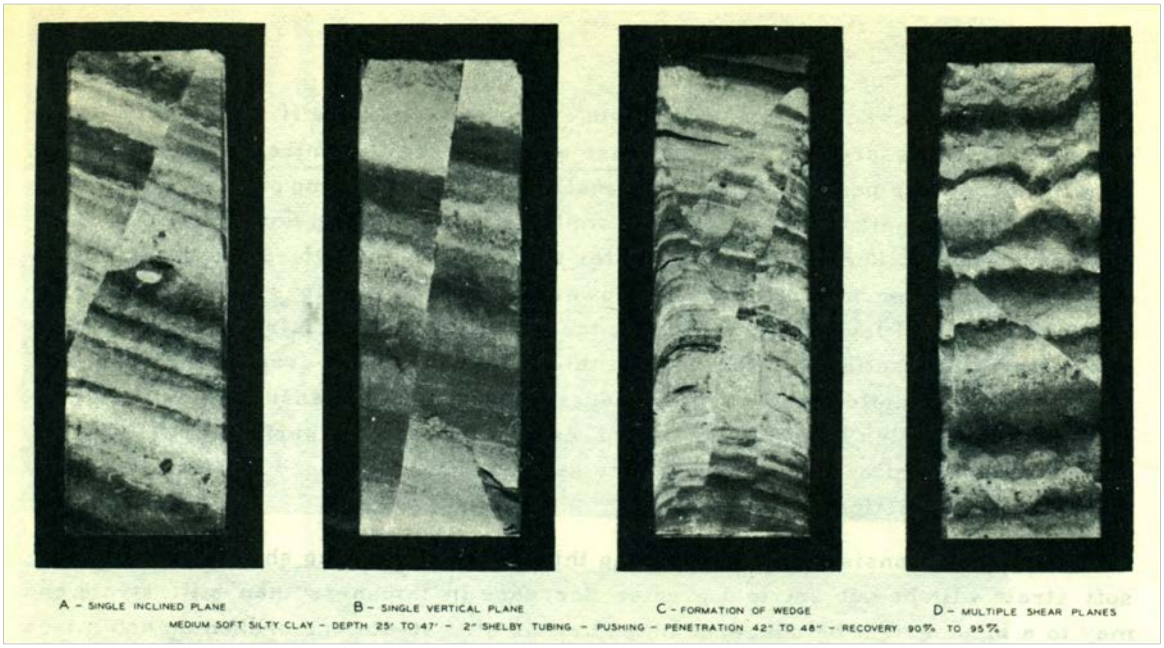

Figure 4-7 Examples of shear failure caused by sampling (Hvorslev 1949). With permission from U.S. Army Engineer Research & Development Center (ERDC).

A remarkable series of photographs is presented by Hvorslev (1965) that shows possible distortions as the soil enters the tube and is affected by inside wall friction (Figure 4-8). If the inside wall friction becomes excessively large, it may prevent any further advancement of the soil into the tube and a permanent bulb of highly disturbed soil will form at (beyond) the end of the sample interval. This disturbed material may show up later in the next sample, depending on the interval spacing, see Figure 4.9. More recent discussions on sample disturbance on various soil properties can be found in the 2006 RM Hardy Address (Graham 2006) with reference to Baligh et al (1987) and Vaughan et al (1993).

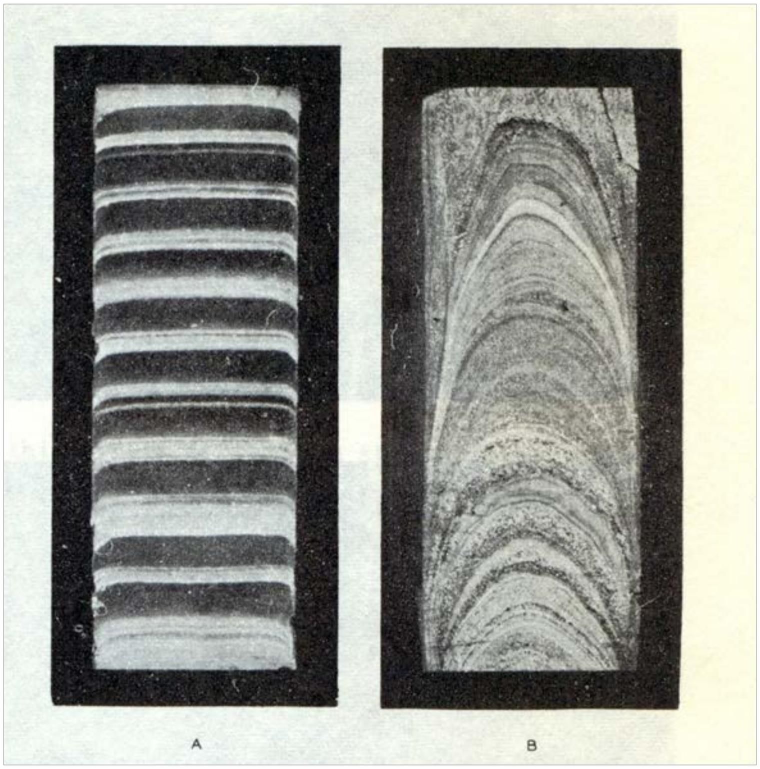

Figure 4-8 Drag and distortion from friction between inside of tube and soil. *Sample A is a varved clay. Sample B is a sandy and silty clay (Hvorslev, 1949).* With permission from U.S. Army Engineer Research & Development Center (ERDC).

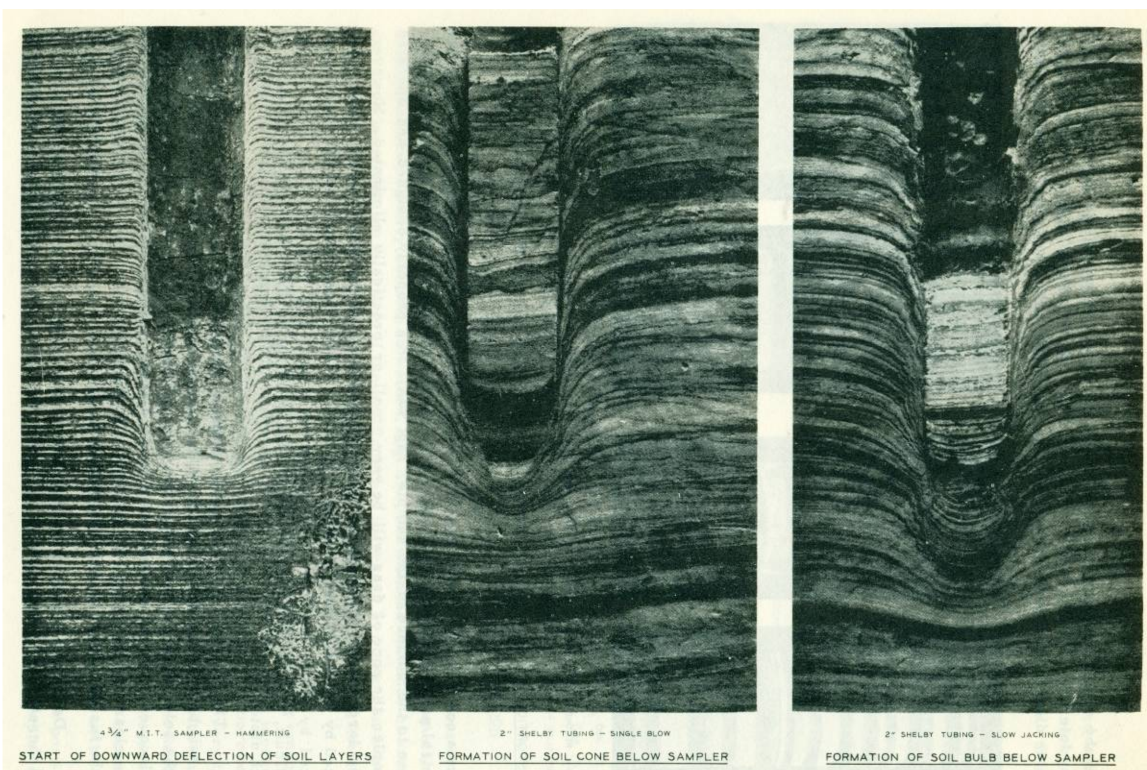

Figure 4-9 Development of soil bulb of highly disturbed soil below Shelby tube starting with downward deflection of soil layers (left), formation of soil cone (middle) and formation of soil bulb (right) (Hvorslev 1949). With permission from U.S. Army Engineer Research & Development Center (ERDC).

The effects of inside wall friction can be reduced by lubricating the inside of the Shelby tube before it is used and by a slight reduction in the diameter of the cutting edge by rolling it inwards to increase clearance between the soil and tube. The first approach is commonly used in Winnipeg but oil must be only very lightly applied to minimize the potential for sample contamination that could affect the diffuse double layer (DDL) and interparticle behaviour. The Author has used the second approach on occasion (in particular for soft samples) but it is common practice at the University of Belfast and the Royal Roads Military College in Kingston Ontario and is recommended for 100 mm diameter Shelby tubes at the University of Manitoba (personal communication J. Graham). Another ingenious method using sliding steel foils was developed by the Swedish Geotechnical Institute; however, there is no information to suggest this method has ever been tried locally. Other approaches involve using what is known as a 'piston sampler' which provides continuous contact with the top of the specimen during extraction. Piston sampling assists recovery in very soft clays and was used briefly in Winnipeg in about 1980 for sampling using 100 mm Shelby tubes.

Clearly, there is much more science behind pushing a Shelby tube and collecting a representative soil than one might think. With this in mind, it is important to be as careful as possible in minimizing these effects when sampling and to take note of any inherent visible effects when examining extruded samples. After inserting the sampling tube into the clay, remember to allow the sample to rest before attempting to withdraw the tube and retrieve the sample. The delay allows the clay swell laterally against the inside wall of the Shelby tube. Increasing the available friction between the soil sample and the inside of the tube increases the likelihood of full recovery.

4.6 *IN SITU* TESTING

In situ testing involves inserting instruments into the soil to determine engineering properties such as undrained shear strength, stiffness, and hydraulic conductivity. There has been limited local use of the pressuremeter, cone penetrometer and flat plate dilatometer in the Winnipeg region to measure compressibility, stiffness and degree of overconsolidation. Because of their limited use, the author has chosen not to discuss these test methods and results in this thesis. More commonly used *in situ* test methods include the field vane (FV) test and the Standard Penetration test (SPT) to directly and indirectly measure undrained shear strength.

4.6.1 Vane Shear Test

The vane is particularly useful in measuring the strength of soft and compressible cohesive soils where sampling may be difficult or where samples cannot be tested accurately in a soils laboratory. Field vane tests should not be done in sands or silts, which tend to be dilative and produce estimates of strength that are much too high.

In its simplest form, the vane may be on the end of a hand pushed rod with a torque wrench at the top of the rod used to measure to indirectly measure shear strength. This apparatus is particularly useful in hand augered test holes. More sophisticated devices and methods are available. For example, the Nilcon Vane uses slip couplers, crank mechanisms, and electrical control boxes to apply and measure torque (Geotech Inc. of Sweden). It may be used in open boreholes or at the end of a hollow stem auger hole.

In all cases, care must be taken to maintain a slow rate of shearing, preferably 0.05 to 0.2 degrees/sec or 3 to 12 degrees per minute (ASTM, D2573). Depending on the soil consistency, this will result in a test time from 2 minutes in firm to stiff clay, to 10 minutes or greater in soft clay (a stiffer cohesive soil will fail at much lower deformation or strain than a softer clay and the test durations are therefore shorter). More rapid testing (which is unfortunately often the tendency in the field), can result in significantly higher measured shear strengths that can lead to less conservative results in design. The peak strength test should be followed by the measurements of a remoulded strength, done immediately after 5 to 10 rapid revolutions of the vane. The ratio of peak to remoulded strength is defined as the Sensitivity (S_t) of the clay. This typically ranges from 2 to 4 for Winnipeg clays (Kjartanson, 1983) and indicates low sensitivity.

It is generally good practice to apply a correction factor to the measured shear strength for design purposes to take account of the plasticity of the clay. The original presentation of correction factors by (Bjerrum 1972) was based on re-analysis of a series of compacted fills that failed shortly after construction. Figure 4-10 shows a proposed alternative method presented by Chandler (1988). For example, the correction factor corresponding to Winnipeg clays with plasticity indices of 40 to 75 would be 0.9 and 0.85 respectively for a 10 minute test, where t_f in the figure is in minutes. For most lacustrine clays with medium plasticity, a correction factor of 0.85 appears to provide a reasonable correlation with laboratory test methods.

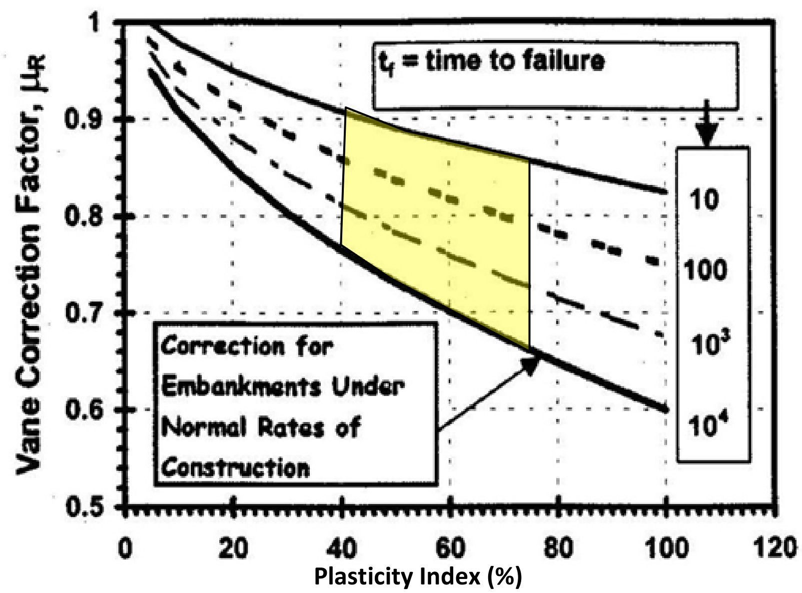

Figure 4-10 Proposed correction factor for raw field vane data (modified from Chandler 1988). *The approximate range of I_P for Winnipeg Clays has been highlighted.* Adapted, with permission, from STP1014-Vane Shear Strength Testing in Soils: Field and Laboratory Studies, copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428.

4.6.2 Standard Penetration Test

Because of the nature of the test, the Standard Penetration Test (SPT) should not be used to measure consistency in the lacustrine clays and silts; local practitioners prefer laboratory testing on Shelby tube samples to determine undrained shear strengths. Empirical relationships that use SPT results relate mostly to sands and gravels and are unreliable in clays.

Standard Penetration testing is sometimes used in the granular tills to obtain an indication of relative density and obtain samples for moisture content determination. Care should be taken to ensure the driving shoe is in good condition and the full drop height of the hammer is utilized otherwise, both will result in erroneously high and therefore nonconservative results. Automatic drop hammers should be considered essential. If upon examination of the sample, it appears that gravel, cobble or boulder may have restricted penetration, the test results should be used with caution or omitted. If SPT blow counts (N) are the only available information, a multiplication factor of 6 provides a reasonable conversion of N values to undrained shear strength in kPa for clay soils. This correlation should be used with caution and is not recommended as good practice. Readers are reminded that correction factors may be needed for the effects of ground water on effective stresses and therefore on correlations between blow-count N and bearing capacity.

4.7 GROUNDWATER MONITORING

Geotechnical investigations often include a groundwater evaluation to determine the depth to the ***water table*** (the level within the soil to which water will rise in an observation well. Technically, it is the level at which the pressure in the pore water is atmospheric. Investigations also examine ***piezometric head*** (the level to which water will rise in a confined aquifer), groundwater flows, yield and quality. The water table can be measured with a simple observation well which will also facilitate collection of samples. In an unconfined aquifer, the piezometric level is coincident with the water table. A piezometer is used to measure static liquid pressure or the piezometric head of groundwater, also known as ***pore water pressure***. A piezometer can be a Casagrande tip, pneumatic tip, or vibrating wire transducer. For seasonal changes in piezometric head, for example in riverbanks, pressure transducers or vibrating wire transducers should be considered necessary. They also permit long-term and remote sensing of pore water pressures.

Ground water conditions are complex. They vary with elevation and with time. It is generally unlikely that a site will have only one water table that can be represented by a hydrostatic pressure that increases linearly with increasing depth, such as would be expected, for example, in an open body of water. In the Winnipeg region, perched groundwater levels can be expected in the shallow silt layers. Piezometric levels in the bedrock, till, and the bottom of the clay layer may be lower or higher than that measured nearer the ground surface, resulting in downward or upward seepage gradients. It may therefore be necessary to install a number of vertically separated piezometers to determine groundwater gradients. These may be important considerations in analysing seepage, deep excavations, and slope stability.

Care needs to be taken about how groundwater monitoring information is presented on a borehole log. If a groundwater level or piezometric elevation is shown, the date when the reading was taken must be provided – levels and elevations typically change with time. The use of different symbols to represent levels recorded immediately after drilling, short term levels, or those which represent stabilized elevations is a good way to illustrate the information, provided the definition of the symbols is included in the borehole log package. Because figures may be used in several places in reports and at different times, symbols should be included on every figure, or at least in every figure caption. Where appropriate, the geotechnical report should also indicate that the groundwater levels reported may change seasonally, after heavy precipitation, or as a result of construction activities.

5 DESIGN METHODS

In the early days of our profession, most civil engineering designs were based solely on experience, judgement and general sets of rules. Today, there are two main philosophies for Civil Engineering design; ***Working Stress Design*** (WSD)[11] and ***Limit States Design*** (LSD). With WSD, a structure is designed by considering its stresses in a "working" condition. A Limit State can be considered a condition beyond which a structure or foundation will no longer fulfill the function for which it was designed. With LSD, the structure is designed by considering the stresses at both the:

- ***Ultimate Limit State*** (ULS) or the situation where the structure collapses, and the
- ***Serviceability Limit State*** (SLS) or the situation where cracks appear or the settlement is unacceptable but the structure has not collapsed.

The principal difference between WSD and LSD methods is how uncertainty is accounted for. For reasons that will be discussed later, the current trend in civil engineering is towards the use Limit States Design. While its application in some areas of geotechnical design is straightforward, in other areas, its application is unclear or the lack of performance data makes its application challenging. This chapter reviews some of these difficulties.

5.1 WORKING STRESS DESIGN

Many structures are still designed based on Working Stress Design. The basic premise of WSD is that the actual stresses must be less than or equal to the allowable stresses. The general relationship for WSD takes on the following form:

[11] Also known as Allowable Stress Design (ASD).

$$\frac{R_n}{F} \geq \sum Q \qquad \text{Eq. 5-1}$$

where: R_n = Nominal (ultimate) geotechnical resistance

F = Safety Factor (SF)[12]

$\sum Q$ = Summation of unfactored force effects (loads)

The WSD method is based on the premise that sound design requires the absence of failure and satisfactory performance in terms of deformations, etc. Although simple in principle, this design method does not account very well for variability in material properties, for example shear strength, or risk (at least based on reliability theory). In WSD, there is no consideration that different types of loads have different levels of uncertainty; dead and live loads are all treated equally. Design loads are usually selected from a specification or design code. The safety factor is applied to the resistance side of the equation only and the load side is not factored. The limitations of this approach can be seen in the graphical representation of the safety factor in Figure 5-1, where the values of Q and R_n are assumed to be unique and therefore have a probability of occurrence of 1.0. It is well known however, that significant variability in these values can be expected, in particular for natural materials such as soil and rock.

In WSD, the selected safety factor may rely on conventional practice, experience, or design codes, and can vary considerably depending on the nature of the problem. For example, a safety factor of 1.3 to 1.5 may be used for earthworks (for example slopes) and a safety factor of 2.5 to 3.0 for foundation design. Safety factors less than 2.5 are sometimes used for temporary works. One might ask why the safety factor for building

[12] Other common variations include "factor of safety, FS, or FOS".

foundations is so much higher than for slopes, unless of course we are accepting different levels of risk. As it turns out, a safety factor of 3.0 for foundations is assumed to limit the settlement of a structure to a tolerable level (say 25mm) based on the linear soil behaviour of most (stiff) soils at stress levels less than 1/3 of the ultimate capacity (Atkinson 2007). Interestingly, the answer to this question suggests there is in fact an implied resemblance between WSD and LSD methods.

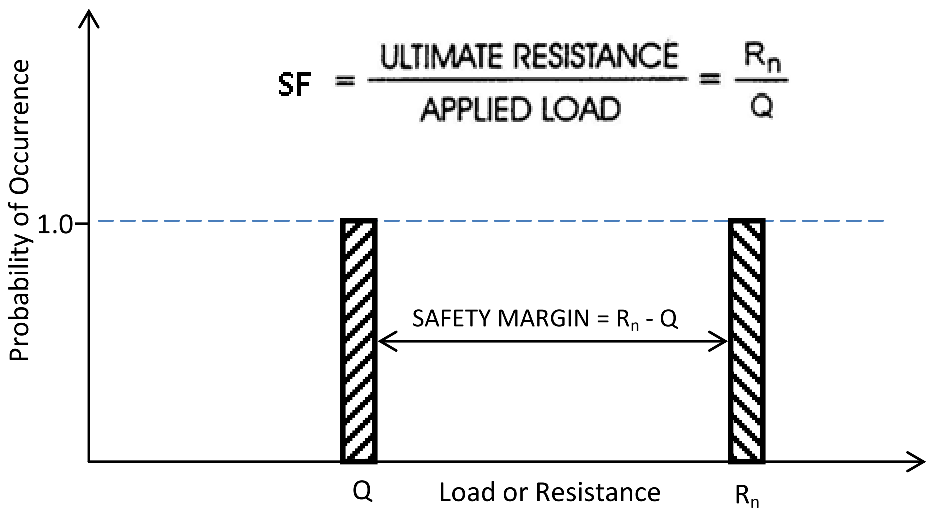

Figure 5-1 Safety Factor (modified from FHWA Manual)

5.2 LIMIT STATES DESIGN

In the mid-1950s, the concept of partial factors was introduced into geotechnical design by Taylor (1948) and Brinch Hansen (1956) who applied separate factors to different types of loads and shear strength parameters for soils[13]. The philosophy of this design method was to yield about the same design outcomes as for the conventional global safety factor approach. The approach was used in Denmark beginning in the 1960s and became

[13] Known as Load and Strength Factor Design.

widely adopted in European practice in 1991 with the introduction of Eurocode 7 - Part 1 in 1990, a general document that provides the principles for geotechnical design within the framework of Limit States Design. Detailed design rules (formulae and charts) are provided in informative "Annexes" - the main reason being the disparity in design models from one country to another.

Meanwhile in North America, an alternative approach was gaining popularity, namely ***Limit States Design (LSD)***, referred to as ***Load and Resistance Factor Design (LRFD)*** in the United States. LSD applies load factors and geotechnical resistance factors based on statistics and a pre-selected probability of failure. An idealized probability distribution is shown on Figure 5-2. The 3rd (1992) edition of the Canadian Foundation Engineering Manual (CFEM) only made mention of LSD, whereas the current (4th) edition (2006) devotes an entire chapter to LSD. This makes the CFEM consistent the current approaches used by the National Building Code of Canada (NBCC), the Canadian Highway Bridge Design Code (CHBDC), the Ontario Bridge Design Code (OHBDC) and the American Association of State Highway and Transportation Officials (AASHTO).

LSD allows both the geotechnical and structural engineers to reach a common goal of achieving an adequate and consistent level of safety as well as minimizing damage and loss of function (CFEM 2006). This is accomplished by using similar design approaches and concepts, which by necessity, encourages dialogue between the two disciplines during both the design and construction stages. There is no doubt that the change from WSD to LSD methods requires a concerted effort on the part of geotechnical engineers, in particular those of a vintage accustomed to WSD.

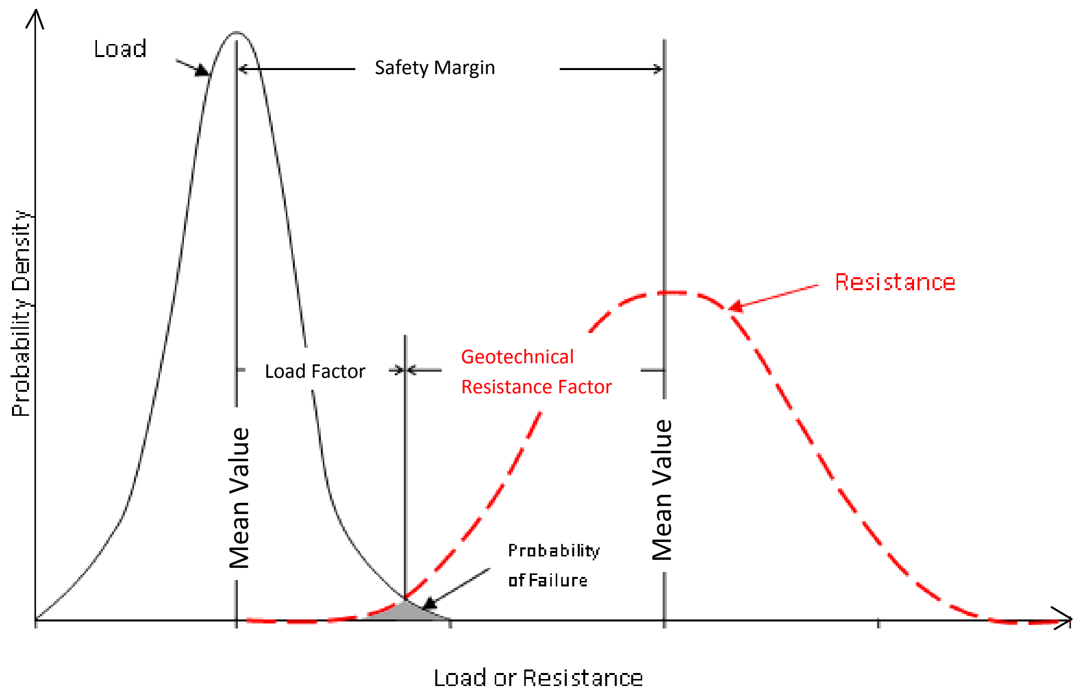

Figure 5-2 Idealized probability distribution using mean values

Over time, the advantages of LSD have become increasingly obvious, in particular as the profession becomes more accustomed with the approach, education on the benefits of LSD is enhanced, additional performance data becomes available, and more design codes adopt the method. LSD for foundations is following the evolution of the structural discipline and is becoming the standard state-of-practice (Becker 1996). It will perhaps be the purely geotechnical problems such as slope stability analysis that will continue, at least for the foreseeable future, to maintain the concept of a global safety factor until at least until such time that their solutions can reliably include safety margins based on statistical data and probability theory. For example, finite element methods have clearly shown that assuming a constant safety factor along the entire slip surface is a major oversimplification as to what actually happens at failure and the spatial variability of materials can be taken into consideration using probabilistic analysis (Van Helden 2013).

From a geotechnical perspective, we can think of LSD as a method that provides a safe design by ensuring that the resistance of the soil (say the bearing capacity) is greater than or equal to effect of the applied loads, that is: ***Geotechnical Resistance* $\geq$ *Effect of Loads*** (Figure 5-2). In applying this basic concept, it is important to understand that both sides of this inequality be evaluated for the same condition. For example, the load applied to a foundation should be in compressive stresses if being compared to the bearing resistance of the soil. When any particular loading condition reaches its Limit State, failure will occur, either as an ultimate limit state or as a serviceability limit state.

A structure must first be designed to satisfy the load carrying capacity requirement or ultimate limit state. This is followed by a check on the serviceability limit state using the loads determined from the ULS. Most structures that satisfy the ULS also satisfy SLS, however exceptions do exist, in particular where:

- High resistance factors are used to determine the geotechnical resistance,
- Foundations are constructed on soils that are settlement prone, or
- Settlement tolerances are very small.

5.2.1 Ultimate Limit State

The Ultimate Limit State is sometimes referred to as the Strength Limit State, a term which unfortunately may be confused with the acronym for Serviceability Limit State (SLS). In the most basic terms, it requires that the factored geotechnical resistance is equal to or greater than the factored structural load. In North America, the geotechnical capacity is determined using a ***Factored Resistance Approach*****;** that is the nominal geotechnical resistance R_n is factored down to the design geotechnical capacity (factored geotechnical resistance) using a resistance factor φ. It is important to note that in Europe, a ***Factored Strength Approach*** may be used whereby the design geotechnical capacity is calculated using ***Partial Reduction Factors*** applied to the geotechnical parameters and that this method may not necessarily produce the same results as the factored resistance approach.

In its simplest form, the basic ULS equation is:

$$R_r = \varphi R_n \geq Q \qquad \text{Eq. 5-2}$$

where: R_r = Factored Geotechnical Resistance

φ = Statistically based Resistance Factor which is generally less than 1

R_n = Nominal Resistance (*i.e.* ultimate capacity)

Q = Factored Structural Load

In North America, the relationship for the Strength Limit State takes on the following more complete form shown by Eq. 5-3.

$$\varphi R_n \geq \sum \eta_i \gamma_i Q_i \qquad \text{Eq. 5-3}$$

where: η_i = Load multiplier to account for effects of ductility, redundancy and operational importance

γ_i = Statistically based load factor which is generally greater than 1

Q_i = ***Unfactored*** Nominal Load

It is important to note that Eq. 5-3 may also use the terms α in place of γ and S_{ni} in place of $Q_{i.}$ - this is common in Canadian practice, for example in the CFEM, (2006). Each potential limit state must be evaluated separately, for example, bearing capacity and overturning. ULS conditions are then checked using separate and unique partial factors on loads (greater than 1) and on the ultimate or nominal resistance (less than 1). The general form of the strength limit state equation can be seen graphically in Figure 5-3.

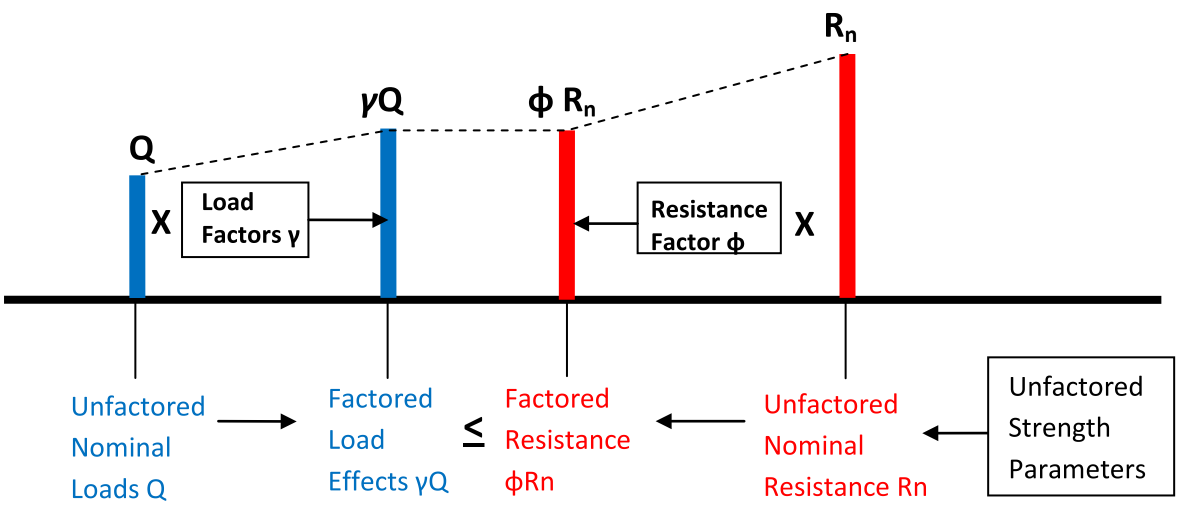

Figure 5-3 LSD approach for the ULS (modified from FHWA Manual)

Load and Resistance Factors

The load factors (γ), load combinations and geotechnical resistance factors (φ) are specified by the various codes including AASHTO, CHBDC and the NBCC or may be found in manuals for state-of-practice, for example, the CFEM (2008). Geotechnical engineers need to be familiar with the load side of the equation as it is important to recognize that the resistance and load factors are interrelated.

Geotechnical resistance factors (φ) typically range from 0.3 to 0.65 to account for uncertainties including:

- The variability in soil and rock properties,
- The structure class, e.g. foundation type or retaining structure,
- The reliability (accuracy) of equations used for predicting resistance,
- Quality of workmanship and quality control programs,
- The reliability of the measurements of material properties,
- Soil behaviour,
- Effects of proposed construction on design,
- The extent of the soil exploration program (site characterization), and
- The consequences of failure.

Geotechnical resistance factors are calibrated based on judgement, other methods or codes such as WSD, reliability theory, or a combination of approaches with the intent of achieving the desired level of safety (Becker 1996). Calibration based on judgement is based on successful previous performance and may lead to unnecessary conservatism. Calibration by fitting with other methods such as WSD involves using resistance factors that would result in the same physical dimension of say a foundation unit such as a footing or pile designed using WSD. Calibration with WSD is generally only used where there is insufficient statistical data to apply reliability theory but has the added advantage of not resulting in a radically different design than the old (WSD) method. For example, a resistance factor can be calibrated to WSD as follows:

1. Divide the LSD Eq. 5-3 with WSD Eq. 5-1 (assuming η_i in Eq. 5-3 = 1)

From which:

$$\phi \geq \frac{\sum \gamma_i Q_i}{F \times \sum Q_i}$$

And therefore:

$$\frac{\phi R_n}{R_n} \geq \frac{\sum \gamma_i Q_i}{F \times \sum Q_i} \qquad \text{(Eq. 5-4)}$$

2. If the total load consists only of dead load (Q_D) plus live load (Q_L), then Eq. 5-4 becomes:

$$\phi = \frac{\gamma_D Q_D + \gamma_L Q_L}{F\left(Q_D + Q_L\right)} \qquad \text{(Eq. 5-5)}$$

3. Dividing both the numerator and denominator by Q_L, Eq. 5-5 becomes:

$$\phi = \frac{\gamma_D \left(Q_D / Q_L\right) + \gamma_L}{F\left(Q_D / Q_L + 1\right)} \qquad \text{(Eq. 5-6)}$$

There are a large variety of dead and live load ratios on which the resistance factors calculated using Eq. 5-6 will depend upon. Figure 5-4 shows the relationship between a conventional WSD safety factor and various dead load and live load ratios for a dead load factor $\gamma_D = 1.25$ and a live load factor $\gamma_L = 1.75$ (within the range specified by AASHTO). For example, a safety factor of 2.5 and 3.0 in WSD is equivalent to a resistance factor of about 0.55 and 0.45 respectively in LSD. Note that in WSD, selecting a value F = 2.5 for a foundation often implies an acceptable performance in terms of both safety and settlement. In LSD, stability and settlements must be examined separately in terms of ULS and SLS respectively: An acceptable ULS design does not necessarily imply an acceptable SLS design.

Calibration using reliability theory utilizes three levels of probabilistic design. The fully probabilistic method (Level III) is the most complex, requiring an understanding of the probability distribution for each random variable and correlations between the variables.

Levels I and II utilize simpler statistical characteristics such as the mean and standard deviation to describe the probability distributions. It is also assumed that the load (Q) and the resistance (R) are independent random variables (that is, events related to one are independent of the other).

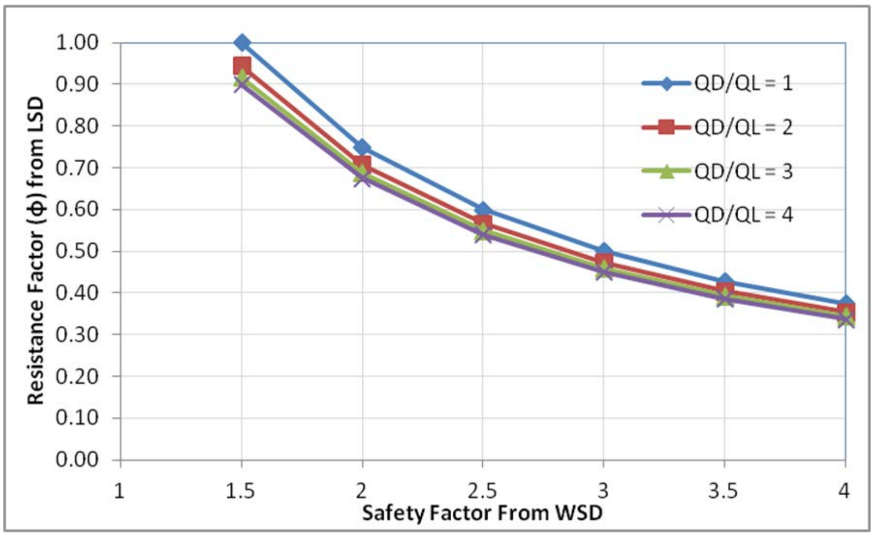

Figure 5-4 Resistance Factors vs. SF for Load Factors γ_D of 1.25 and γ_L of 1.75

There are several areas of geotechnical design that have not been calibrated for LSD; for example, static slope stability analysis. While most practitioners still think of analytical solutions for these problems in terms of a global safety factor, such problems can be also be solved in terms of a quasi-LSD approach where the load factor γ is set at 1.0 and the resistance factor (φ) is the inverse of the safety factor (1/F). With this approach, the probabilistic solution will have the same margin of safety (or probability of failure) as would be arrived at with WSD. This is illustrated by re-writing Eq. 5-2 as follows:

$$\frac{Rn}{Q} = \frac{Resisting\ Force}{Driving\ Force} = \mathrm{SF} \geq \frac{1}{\varphi} \qquad \text{Eq. 5.7}$$

where: R_n = Nominal Resistance (*i.e.* ultimate capacity)

Q = Factored Load (with load factor γ = 1.0)

SF = Safety Factor

φ = Resistance Factor

Nominal Strength

Current design codes generally only specify resistance factors; there is no guidance provided on the determination of geotechnical properties, in particular the ***nominal strength***. The nominal strength of structural materials such as steel is the specified tensile yield strength. However, the nominal resistance of soils is determined differently and is commonly taken as the ***ultimate strength*** of the soil derived from suitable testing methods or empirical relationships. The nominal value is also referred to as the ***characteristic value*** (f_k) by some practitioners (Becker 1996).

Since there is no codified method of determining the nominal resistance, it may be selected based on a variety of methods that will include an evaluation of appropriate *in situ* testing or laboratory testing which takes into account sample disturbance, test methods, stress-path dependency, etc. As such, the selected value for nominal resistance is highly dependent upon the experience and judgement of the geotechnical engineer interpreting the data, and therefore may vary from one engineer to another, in much the same way as with WSD. This important consideration clearly demonstrates that engineering judgement does still play an important role in the LSD method. It should be emphasized however, that engineering judgement should always be applied carefully and never as a method to account for insufficient geotechnical information (SCDOT 2008).

It is also important to develop unambiguous and consistent methods for determining (or selecting) the nominal resistance (Becker 1996). Arbitrarily selecting a conservative nominal resistance may invalidate the assumptions used in determining the necessary resistance factor to account for the actual level of uncertainty in assessing the soil properties. The application of a conservative nominal resistance may be justified however, when only limited or highly variable data is available, thereby reducing the risk of poor performance.

Several methods have been developed towards achieving consistent nominal resistances, although none are prescribed. As a result, selection is more often than not based on a geotechnical engineer's "best estimate" of the likely value. If sufficient data exists and statistical methods are used, the nominal (characteristic) value is sometimes interpreted as being the 75% value at a confidence level of 50%, that is 50% of the values lie within 0.7 standard deviations of the mean, as shown on Figure 5-5 (Dahlberg 1993).

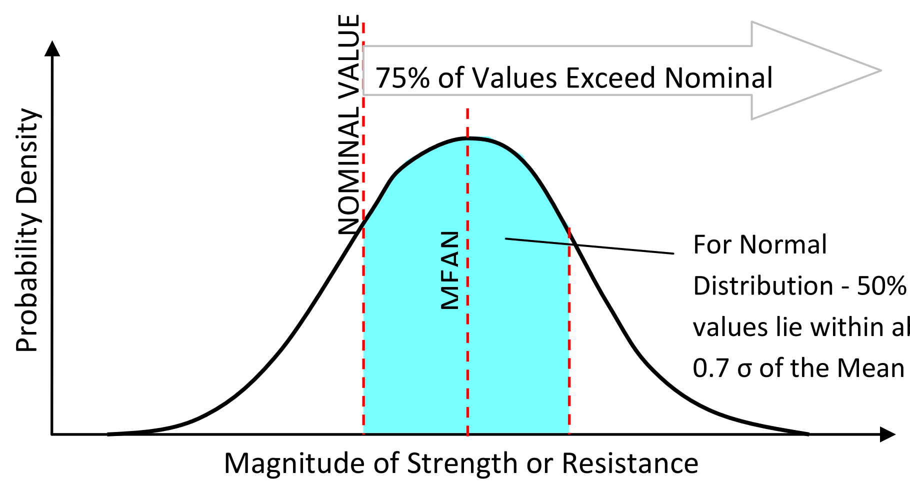

Figure 5-5 Possible Interpretation of Nominal Value (based on Dhalberg,1993)

There is also merit in determining soil (strength) properties consistent with the manner used to determine the resistance factors. In this regard, resistance factors for AASHTO are based on reliability theory, determined using average shear strength properties for various geologic units (SCDOT 2008). Determining resistance factors on the basis of a WSD back-analysis should use the same method for selecting soil strengths in both analyses.

5.2.2 Serviceability Limit State

Although by satisfying the ultimate limit state a structure may be safe to carry the loads it is intended to, it may be still be unable to serve its intended purpose due to excessive deflection or deformation. The Serviceability Limit State (SLS) requires that under serviceability loading conditions, the deflections do not exceed tolerable limits determined by operational, durability, or aesthetic requirements. Serviceability loads are generally a combination of unfactored dead loads plus a reduced live load component. Conditions for the SLS are checked using the serviceability load and unfactored geotechnical properties; essentially a partial factor of 1.0 is used for deformation properties of the soil or rock. It is important to understand that the design serviceability load is not necessarily the same as the working load used in WSD.

There is general agreement that a reduction factor should not be applied to geotechnical parameters used when assessing the serviceability limit state. Rather, a conservative estimate of the mean value obtained from *in situ* or laboratory tests is often used as a characteristic geotechnical value providing that stress level dependency has been properly considered. This is another example of where engineering judgement and experience must factor into a codified design method that otherwise has its roots in reliability theory.

5.3 Application to Local Geotechnical Practice

As specified in the National Building Code of Canada (NBCC), foundations for multi-family and commercial structures in Winnipeg must now be designed using LSD. One of the first challenges to adopting the LSD approach for local practice has been an inevitable comparison between foundations designed with the new method compared with that which would have been designed following the older WSD method. This has resulted in considerable (but meaningful) discussions between geotechnical and structural engineers, particularly in cases where the new design code has resulted in additional foundation elements being required. This is largely due to the prescribed load and resistance factors which may indeed have been calibrated to WSD, but may not necessarily result in a similar foundation design to what would have been obtained using empirical design values that have been developed locally in the past. Design rules for driven precast concrete piles are an example. Chapter 6 of this thesis will discuss this comparison of design methods for deep foundations in more detail.

The selection of resistance factors is not necessarily based on prescribed methods for site characterization, sampling, *in-situ* testing or laboratory testing. As a result, the approach to obtaining geotechnical information for design can change substantially from one geotechnical engineer to another depending on experience, preference, and often budgetary constraints. This variability within industry is inconsistent with the working principle for LSD, whereby soil strength parameters are determined in a specific and consistent manner in order that resistance factors can be held constant. Consider that the NBCC prescribes a resistance factor of 0.5 for the design of shallow foundations regardless of the number and test methods used to measure undrained shear strength. For example, the resistance factor used for design is the same regardless if the determination of nominal soil strength is based on unconfined compression (UC) tests or consolidated undrained (CAU) triaxial tests.

Clearly, a greater degree of uniformity of the practice of site characterization will be required in the future to arrive at nominal values that are repeatable and consistent. Applying resistance factors to poorly defined or incorrect nominal values will render the LSD design approach meaningless. It would be of significant benefit to follow the lead of the Danish Code a degree of uniformity is obtained by formally specifying three classes of site investigations that lead to separate values of resistance factors (Becker 1996).

Local practitioners would also greatly benefit from static or dynamic load tests for deep foundations where nominal capacities could be measured and settlement data obtained to allow more accurate predictions of the serviceability limit state. The author suggests that load tests are needed for cast-in-place friction piles; driven steel and precast concrete piles; and caissons founded in till or bedrock. This is of particular importance when the serviceability limit state rather than the ultimate limit governs the design. While this situation may not be typical, it becomes increasingly likely when higher resistance factors are used to determine the Ultimate Limit State. Moving forward, such testing may become more attractive, given the construction and economic benefits of being able to use higher resistance factors and opportunity for overall design optimization.

6 DEEP FOUNDATIONS

This chapter will deal only with deep foundations which are to support moderately to heavily loaded structures. As such, it will not include detailed discussion of shallow foundations for lightly loaded structures and the well-established environmental effects on shallow foundations due to the expansive properties of Lake Agassiz clay in Winnipeg. These have been described well in a series of publications, notably by Hamilton (1969), Bozozuk (1962), Meyerhof (1965), Kjartanson et al (1983), and the Canadian Foundation Engineering Manual, 4th edition (2006).

A discussion of design practices for deep foundations is considered timely given the recent shift from Working Stress Design (WSD) towards Limit States Design (LSD). It seems that much of the design methodology developed by early practitioners such as R.M. Hardy may not be well understood or may have been forgotten by today's practitioners. While the design and performance of deep foundations in the Winnipeg region based on WSD methods has proved to be successful, the capacities associated with this method may not be the same as those determined using LSD methods. If the geotechnical engineer is to fully understand the reasons for this discrepancy and rationally consider possible methods to arrive at similar (successful) foundation designs while maintaining Code requirements, an understanding of the historical development of local design methods is valuable, and perhaps necessary.

The intent of this chapter is therefore to compare the design methods, and where appropriate, suggest applications of LSD methods which may produce designs more comparable with those expected from WSD methods. In this regard, it is not the Author's intent to question or modify codified design methods and parameters but to look more carefully at the basis for which traditional allowable foundation capacities have been adopted and how these values (in particular nominal capacities) relate to both the ultimate and service limit states.

6.1 HISTORICAL DESIGN METHODS

In 1926, A.W. Fosness, a design engineer with Carter-Halls Aldinger Company, authored a technical paper titled "Foundations in the Winnipeg District" which was presented before the Winnipeg Branch of the Engineering Institute of Canada (Fosness 1926). This landmark paper discussed the engineering properties of the Winnipeg clay and till as they were understood at the time, allowable bearing pressures for shallow foundations, floating foundations, pile foundations, foundations on clay, foundations along riverbanks and the relative costs of various foundation types. During the summer of 1937, a technical committee headed by Professor A.E. MacDonald with assistance from Professor W.D. Riddell was formed to study the problem of soil conditions in the Winnipeg district with particular emphasis on foundations (MacDonald 1937). These accounts illustrate the pioneering spirit that led the way to modern design and construction practice in the city of Winnipeg.

Early in the 1900s, bearing capacities of 287 kPa (3 tsf) for warehouses and 239 kPa ($2^1/_2$ tsf) for office buildings were considered appropriate when founded on "blue clay"[14]. These heavily loaded structures often experienced excessive settlement and in an attempt to reduce these settlements, the allowable bearing capacities shown in Table 6-1 were adopted in the "new" Building Code. The lower values for office buildings and

[14] The term blue clay is a historical account generally referring to the grey clay layer below the upper brown clay.

apartments recognized the problems with interior finish and plumbing that could be expected with greater settlement magnitudes.

Table 6-1 Allowable Bearing Capacities (c.1926)

Soil Type at Foundation Level	Allowable Bearing Capacity in Tons Per Square Foot (tsf)	
	Warehouses, manufacturing buildings, etc.	Office buildings, apartments, etc.
Firm blue clay with no underlying strata of yellow clay[15]	2	$1\frac{1}{2}$
Mixed clay - moderately dry	1	$\frac{3}{4}$
Soft yellow clay and silt	$\frac{1}{2}$	$\frac{3}{8}$

Early in the 20th century, several large and heavily loaded structures had been erected in the central business section of Winnipeg, notably the Hudson's Bay Company Store, Eaton's Store, the Manitoba Cold Storage Building and the Legislative Building. All of these structures had caissons advanced to the limestone bedrock which ranged from 15.7 to 22.3 m ($51\frac{1}{2}$ to 73 ft) below prairie grade at the Hudson's Bay Building and Manitoba Cold Storage Building respectively. Of particular concern to contractors of the day was the occurrence of vertical fissures[16] in the limestone running in irregular lines which were often filled with shattered rock or sand. In those days, caissons were 1.2 m (4 ft) diameter holes, dug by hand, with vertical wood staves and split steel shoring rings, wedged tightly

[15] The term yellow clay is a historical reference to silt or clayey silt layers.
[16] The term fissure is a historical reference to a fracture or vertical joint in the bedrock.

in place. These were also referred to as "Chicago Wells" - an example of underpinning the Transcona Grain Elevator in 1913 is shown on Figure 6-1.

Figure 6-1 Sinking Chicago Well at the Transcona Grain Elevator
(photo courtesy of B. Parrish Sr.)

Concrete caissons were typically designed with a permissible bearing pressure of 2870 kPa (30 tsf) when founded on rock and 718 kPa ($7^1/_2$ tsf) when founded on dense till, with the requirement that it be belled to twice its diameter. Interestingly, owners were allowed to reduce the live load value by 25% to encourage the use of caissons over more conventional floating (mat) foundations.

Perhaps one of the most interesting historical accounts of sinking caissons relates to the construction of the Hudson's Bay Store on the southeast corner of Portage Avenue and Memorial Boulevard where more troubles with "crevices or rock pockets [17] " were

[17] Historical terms referring to fissures in the bedrock filled with sharp-edged shattered limestone or sand.

encountered than on any other work in the city (Figure 6-2). One crevice ran across the southwest corner of the building where it interfered with only one caisson. Six caissons however, coincided with another crevice in the bedrock on the northeast side of the building within which pockets of fine white and "jet black" water bearing sand were encountered. While trying to dewater the caisson with several large pumps, a cavity some 20 m^3 was created under Portage Avenue when sand flowed into the caisson. The caisson was abandoned and another was sunk into the base of the crevice to a depth of 29 m (95 ft) where sand was not evident. This caisson was left open and served as a sump to allow the sand to be dewatered in the area of the problem caissons and allow them to be advanced in the dry. The abandoned caisson was then concreted, as was the cavity beneath Portage Avenue.

Figure 6-2 Excavation for the Hudson's Bay Store in Winnipeg (c. 1926) *A total of 151 caissons were advanced to the limestone bedrock and no building at the time had had a higher percentage of bad caissons (Fosness 1926)*

Before an understanding of effective stresses had been developed, geotechnical engineers of the day were challenged by the relationship between bearing pressures, footing size and settlement. They began to realize however, that it was differential settlement that was the cause of many structural problems and in this regard, it was considered good practice to proportion every footing according to the load so that every footing would be working at approximately the same bearing pressure (Fosness 1926). This practice is still considered appropriate. Foundations bearing on the "soft brown clay [18] " were discouraged, as was placing foundations at different elevations. The effects of moisture content changes in the clay were also well documented, with one notable example of clay beneath large baking ovens drying to a powder and causing not only settlement of the ovens, but also parts of the building in the vicinity of the ovens. This observation is believed to be the account of an addition to the former Speirs Parnell Bakery located at 666 Elgin Avenue in Winnipeg.

In 1947, the Winnipeg Building Code adopted the allowable bearing capacities from 1926 with minor modifications to the building descriptions; warehouses, manufacturing buildings, etc. became "industrial and commercial buildings" and office buildings, apartments, etc. became "buildings for human habitation". The City of Winnipeg Building Code from 1965 was modelled after the National Building Code of Canada (NBCC). It provided the allowable bearing capacities shown in Table 6-2. Recognizing ongoing technical advances of the day however, the 1965 Code also allowed bearing capacities to be determined based on site investigations, field and laboratory testing (Chakrabandh 1972).

[18] Refers to the silt layer.

Table 6-2 Allowable Soil Pressures From 1965 Winnipeg Building Code

Type and Condition of Soil or Rock		Design Bearing Pressure		
		psf	tsf	kPa
Cohesionless soils: dense sand, dense sand & gravel		6,000	3	288
Cohesive Soils	Firm Silt	1,000	0.5	48
	Soft Silt	500	0.25	24
	Stiff Clay	3,000	1.5	144
	Firm Clay	2,000	1	96
	Soft Clay	1,000	0.5	48
Hard Till or Hardpan		15,000	7.5	718
Limestone Bedrock	Sound	60,000	30	2873
	Soft or Shattered	20,000	10	958

Driven precast-prestressed hexagonal concrete (PPHC) piles have been used in the Winnipeg area since about 1961 and at that time, their design was governed by the 1960 NBCC. This code required that piles be designed using one of the following:

- A load test carried out in accordance with good engineering practice,
- Local practice,
- The end bearing capacity determined by the allowable soil bearing pressure,
- The frictional capacity determined by the frictional resistance of the soil, or
- The Hiley Pile Driving formula.

Subsequent editions of the NBCC in 1965 and 1970 essentially maintained this basis of design. In 1964, the Winnipeg Building By-Law 711 was introduced which deviated from the NBCC in that it tabulated the allowable capacities of driven PPHC piles at 450, 620 and 800 kN (50, 70, and 90 tons) for 300, 350, and 400 mm (12, 14 and 16 in) pile diameters respectively. These "historical" values required certain driving conditions, and the reference to the Hiley formula and end bearing capacity based on the allowable soil (*or rock*) bearing pressure were deleted. By-Law 711 was replaced in 1974 by By-Law 740/74 with no changes to the design of pile foundations. In 1977 however, the Manitoba Building Code was adopted which required that foundation design be based on:

- Applying generally accepted geotechnical and civil engineering principles,
- Local practice,
- A load test, or
- Innovative approaches

All reference to the "historical" pile capacities for PPHC piles was deleted in the 1977 Manitoba Building Code. Based on this deletion, independent pile capacity studies were undertaken by three local consultants in 1978 to develop a "state of the art" approach to the design of PPHC piles in Manitoba. Block McLellan and Associates Ltd. carried out pile load tests at the Con-Force Ltd. plant in Winnipeg, R.M. Hardy and Associates carried out wave equation analysis and Klohn Leonoff Consultants Ltd. undertook a pressuremeter investigation. The results were summarized in the Prestressed Concrete Pile Capacity Study which was published in 1978 (Conforce et al 1978).

Based on the results of this work, it was concluded that the safe load carrying capacity of these piles was significantly greater than the historical values, perhaps by as much as 80% higher depending on subsurface conditions, the pile dimensions and the delivered energy. It was felt that with the exception of certain combinations of these variables, it would be reasonable to assume that an increase on the order of 30% was warranted which would have resulted in the values shown in Table 6-3. Despite the significant findings from this report, local foundation design largely continued using the lower "historical" pile capacities, based largely on successful performance.

Table 6-3 Allowable Capacities For PPHC Piles

Pile Diameter	Historical Winnipeg Building By-law 711		Historical Plus 30%	
	tons	kN	tons	kN
(12 in) 300 mm	50	~ 450	~ 65	~ 600
(14 in) 350 mm	70	~ 625	~ 90	~ 800
(16 in) 400 mm	90	~ 800	~ 120	~ 1050

6.2 Current Foundation Design Methods

6.2.1 Building Foundations

Beginning in 2012, major occupancies, including multi-family and commercial buildings in Winnipeg became regulated by Article 1.3.3.2 - Division A of the Manitoba Building Code. Article 1.3.3.2 states that the 2010 NBCC is adopted as the building code in Manitoba, requiring that Limit States methods be used for the design of all foundation types for these structures, including shallow foundations (for example footings), cast-in-place concrete friction piles, driven PPHC and steel end bearing piles, concrete caissons, etc. In this regard, Commentary K of the NBCC (2010) provides resistance factors to be used to determine factored Ultimate Limit State (ULS) capacities as shown in Table 6-4.

Table 6-4 Resistance Factors For ULS Capacities (NBCC 2010)

Description			Resistance Factor
DEEP FOUNDATIONS	Bearing resistance to axial loads	Semi-empirical based on analysis using laboratory and *in situ* test data	0.4
		Analysis using static load testing results	0.6
		Analysis using dynamic monitoring results	0.5
		Uplift resistance by semi-empirical analysis	0.3
		Uplift resistance using load testing results	0.4
	Horizontal Load Resistance		0.5

It should be pointed out that Commentary K states "*This Commentary provides guidance, compatible with sound engineering practice, for the design of foundations and temporary excavations in accordance with the provisions of Section 4.2, Foundations, of the National Building Code of Canada 2005 (NBC). NBC Subsection 4.1.3 requires the use of limit states design for the design of buildings and their structural components. This Commentary deals with this approach for the design of shallow and deep foundations. The material herein is intended as a first approximation dealing with routine problems of foundation design and construction. Neither this material nor the papers or texts to which it refers should substitute for the experience and judgement of a professional engineer competent in dealing with the complexities of foundation design practice*". Depending on one's interpretation of this statement, it may be justifiable to use engineering judgement and experience to select alternative resistance factors for design.

6.2.2 Bridge Foundations

Bridge foundations are often designed using load and resistance factor design (LRFD) methods (similar to LSD) provided by either the American Association of State Highway and Transportation Officials (AASHTO) or the Canadian Highway Bridge Design Code (CHBDC).

AASHTO

Resistance factors to be used for the determination of factored ULS capacities following AASHTO for driven piles and drilled shafts are summarized in Table 6-5 and 6-6 respectively. Article 10.5.5.2.1 states that the specified resistance factors (Table 6-5) are to be used *"unless regionally specific values or substantial successful experience is available to justify higher values"*. However, AASHTO also defines a site as *"the project site or a portion of it, where the sub-surface conditions can be characterized as geologically similar in terms of sub-surface stratification, i.e., sequence, thickness, and geologic history of the strata, the engineering properties of the strata, and groundwater conditions"*. Based on this definition, it may not be appropriate to use the results from another site to establish the applicable resistance factors, even if the geologic conditions are similar.

Table 6-5 Resistance Factors For ULS Capacities - Driven Piles (AASHTO 2010)

<table>
<tr><th colspan="3">Condition/Resistance Determination Method</th><th>Resistance Factor</th></tr>
<tr><td rowspan="7">Nominal Bearing Resistance of Single Pile - Dynamic Analysis and Static Load Test Methods</td><td colspan="2">Driving criteria established by successful static load tests of at least one pile per site condition and dynamic testing of at least two piles per site condition, but no less than 2% of the production piles</td><td>0.80</td></tr>
<tr><td colspan="2">Driving criteria established by successful static load tests of at least one pile per site condition without dynamic testing</td><td>0.75</td></tr>
<tr><td colspan="2">Driving criteria established by dynamic testing conducted on 100% of production piles</td><td>0.75</td></tr>
<tr><td colspan="2">Driving criteria established by dynamic testing, quality control by dynamic testing of at least two piles per site condition, but no less than 2% of production piles</td><td>0.65</td></tr>
<tr><td colspan="2">Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance</td><td>0.50</td></tr>
<tr><td colspan="2">FHWA-modified Gates dynamic pile formulae (end of drive conditions only)</td><td>0.40</td></tr>
<tr><td colspan="2">Engineering News (as defined by Article 10.7.3.8.5) dynamic pile formulae (end of drive conditions only)</td><td>0.10</td></tr>
<tr><td rowspan="3">Nominal Bearing Resistance of Single Pile - Static Analysis Methods</td><td rowspan="3">Side Resistance and End Bearing: Clay and mixed soil</td><td>α method (Tomlinson, 1987; Skempton, 1951)</td><td>0.35</td></tr>
<tr><td>β method (Esrig & Kirby, 1979; Skempton, 1951)</td><td>0.25</td></tr>
<tr><td>λ method (Vijayvergiya & Focht, 1972; Skempton, 1951)</td><td>0.40</td></tr>
<tr><td>Block Failure</td><td colspan="2">Clay</td><td>0.60</td></tr>
<tr><td rowspan="8">Uplift Resistance of Single Piles</td><td colspan="2">Norlund Method</td><td>0.35</td></tr>
<tr><td colspan="2">α method</td><td>0.25</td></tr>
<tr><td colspan="2">β method</td><td>0.20</td></tr>
<tr><td colspan="2">λ method</td><td>0.30</td></tr>
<tr><td colspan="2">SPT method</td><td>0.25</td></tr>
<tr><td colspan="2">CPT method</td><td>0.40</td></tr>
<tr><td colspan="2">Static load test</td><td>0.60</td></tr>
<tr><td colspan="2">Dynamic test with signal matching</td><td>0.50</td></tr>
</table>

Condition/Resistance Determination Method		Resistance Factor
Group Uplift Resistance	All Soils	0.50
Lateral Geotechnical Resistance of Single Pile or Pile Group	All Soils and Rock	1.0

Table 6-6 Resistance Factors For ULS Capacities - Drilled Shafts (AASHTO 2010)

Method/Soil/Condition			Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts	Side resistance in clay	α method (O'Neil and Reese, 1999)	0.45
	Top resistance in clay	Total stress (O'Neil and Reese, 1999)	0.40
	Side resistance in sand	β method (O'Neil and Reese, 1999)	0.55
	Tip resistance in sand	O'Neil and Reese, 1999	0.50
	Side resistance in IGMs	O'Neil and Reese, 1999	0.60
	Tip resistance in IGMs	O'Neil and Reese, 1999	0.55
	Side resistance in rock	Horvath and Kenney (1979); O'Neil and Reese (1999)	0.55
	Side resistance in rock	Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	CGS (1985); O'Neil and Reese (1999)	0.50
Block Failure	Clay		0.55
Uplift Resistance of Single-Drilled Shafts	Clay	α method (O'Neil and Reese, 1999)	0.35
	Sand	β method (O'Neil and Reese, 1999)	0.45
	Rock	Horvath and Kenney (1979); Carter and Kulhawy (1988)	0.40

Method/Soil/Condition		Resistance Factor
Group Uplift Resistance	Sand and Clay	0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All Materials	1.0
Static Load Test (compression)	All Materials	0.70
Static Load Test (uplift	All Materials	0.60

CHBDC

Resistance factors to be used for the determination of factored ULS capacities following the CHBDC for deep foundations are summarized in Table 6-7.

Table 6-7 Resistance Factors For ULS Capacities - Deep Foundations (CHBDC, 2010)

Application		Resistance Factor
Static Analysis	Compression	0.4
	Tension	0.3
Static Test	Compression	0.6
	Tension	0.4
Dynamic Analysis	Compression	0.4
Dynamic Test	Compression (field measurement & analysis)	0.5
Horizontal Passive Resistance		0.5

Section 6.6.1 of the CHBDC states that "*the factored geotechnical resistance at the ULS of a deep or shallow foundation shall be the ultimate geotechnical resistance multiplied by the relevant resistance factor unless a higher value is approved*". A definition of

"approved" is not provided: however it is believed to be associated with acceptance of a higher resistance factor by the agency responsible for the bridge construction.

6.2.3 The application of LSD For Deep Foundations in the Winnipeg Region

By necessity, local practitioners have become increasingly familiar in recent years with the application of LSD in the design of foundations. Inevitably, comparisons have been made between foundations designed using this method and those designed using the former WSD by structural designers who are now faced with potential changes in the overall size of foundation units. Where the SLS capacity governs the design, there may be little difference in the foundation design based on WSD and LSD methods. However, depending on the nominal capacity and the resistance factor applied, the ULS may govern the design, resulting in an increased foundation size compared to what would be determined using WSD design methods. This is most likely to occur where a low resistance factor is applied, for example, 0.4 or less depending on the applicable design code.

Given the incentive to use static and dynamic load testing of installed piles and the resulting ability to apply higher resistance factors, it is likely that information will soon become available on measured capacities for various till and bedrock conditions and driving energies. Recent studies at the University of Manitoba (Belbas 2013) point in this direction. The results of dynamic testing can be used to calibrate wave equation analysis and allow more accurate determination of nominal capacities and load-displacement relationships. This chapter is intended to provide guidance in this regard, keeping in mind that codified values, for example geotechnical resistance factors, should only be altered on the basis of experience and sound judgement and never to simply arrive at compatibility with WSD methods. This approach will be examined in more detail in the following section.

6.2.4 Pre-stressed Precast Hexagonal Concrete Piles

In Winnipeg, pre-stressed precast hexagonal concrete (PPHC) piles are most often driven into the dense till to practical refusal to carry moderate to heavy loads. They are generally considered to be end bearing piles. In lieu of PPCH piles, cast-in-place concrete friction piles are commonly used to carry moderate loads and will be discussed in subsequent sections of this chapter. One approach to estimate the SLS value for a PPHC pile may be to equate it to the allowable capacity from WSD methods, for example, 800 kN for a 450 mm diameter pile[19], with a stated (expected) maximum settlement in the order of 20 mm. This may be considered a reasonable approach in consideration of many years of successful performance at this loading but at the same time seems logically flawed in relation to the fundamental principles of LSD. The inherent difficulty with evaluating the SLS from WSD clearly demonstrates the need for load testing to assess the load deformation properties of deep foundation units in Winnipeg.

While the safety factor associated with the allowable WSD pile capacity is not known with certainty, it is often assumed to be 2.5. With the safety factor removed, the nominal pile capacity for a 450 mm diameter PPCH pile would be 2000 kN. Factored ULS capacities can then be determined by multiplying the nominal capacity by the appropriate resistance factor. For example, with a resistance factor[20] of 0.4 the factored ULS capacity

[19] Refer to Table 6-3
[20] Refer to NBCC values in Table 6-4

would be 800 kN. The ULS and SLS values in this example are identical in part because the resistance factor has been calibrated with respect to the WSD, that is, a resistance factor of 0.4 is equivalent to a safety factor of 2.5.

Winnipeg practitioners may choose the above rationale for LSD for a variety of reasons that include geologic variability in the till, uncertainty in pile driving (delivered) energy, the lack of load test data (to failure) and the understandable tendency to incorporate many years of successful performance, particularly with regard to expected settlements. However, the resulting increase in the size of foundation systems (in some cases) for foundation systems designed using LSD suggests that results of the 1978 Precast Prestressed Concrete Pile Capacity study should at least be considered in determining design capacities. For example, if the "historical" allowable pile capacity is increased by 30% as suggested by the study, the revised SLS capacity would be 1050 kN. Further justification for this approach lies in settlements measured in the field testing program in the order of 6 mm (including elastic compression) at loads of about 1300 kN (Conforce et al 1978). By removing the assumed safety factor of 2.5, the revised nominal capacity would be 2625 kN and the factored ULS capacity ($\phi = 0.4$) would be 1050 kN. The capacities using this alternative approach are summarized in Table 6-5 for resistance factors of 0.4, 0.5 and 0.6 from the NBCC.

Table 6-5 Alternative Capacities For PPCH Piles in the Winnipeg Area[1]

Pile Dia. (mm)	Pile Capacities (kN)						
	Historical	***Historical* plus 30%**	SLS	Nominal	Factored ULS		
					Resistance Factor		
					0.4	***0.5***	***0.6***
300	**445**	579	580	1448	***580***	***720***	***870***
350	**625**	812	810	2030	***810***	***1020***	***1220***
400	**800**	1040	1040	2600	***1040***	***1300***	***1560***

[1] Driven with a hammer of sufficient energy to practical refusal in dense till

The 1978 study concluded that *"the condition most amenable to higher pile capacity is the presence of dense to very dense glacial till which is boulder free"*. There are therefore important issues to be considered if the alternative pile capacities from the 1978 study are to be used in design. One is the condition of the till at the original (1978) test site compared with its condition at another part of the city where the pile foundation is to be constructed. In this regard, the consistency of the till is known to be variable throughout the Winnipeg region and therefore, the conditions associated with the 1978 study cannot necessarily be extrapolated to other locations. With this in mind, the Author proposes that using higher alternative capacities should be considered only on the basis of detailed site investigations where the presence of favourable geologic conditions can be confirmed. A more defensible approach may be to use historical values when assigning preliminary SLS capacities and the alternative (higher) nominal values for the determination of factored ULS capacities (bolded in Table 6-5). The intent of this approach would be to maintain expected performance while recognizing that the ultimate load at failure is considerably higher than would be predicted using the historical values. It must be recognized however, that should the estimated settlement exceed what is tolerable for the structure (under the factored loads assumed for serviceability), it may be necessary to revise the SLS values for design.

6.2.5 Cast-in-Place Concrete Friction Piles

Cast-in-place (CIP) concrete friction piles are commonly used in the Winnipeg region to carry moderate loads. In principle, concrete friction piles develop their resistance through a combination of shaft friction and end bearing, although shaft friction is usually dominant. Two methods of pile analysis are generally used internationally; the alpha method (α-method) using total stress methods, and the beta method (β-method) using effective stress principles. A third effective stress method, the Lambda method is less commonly used and will not be discussed. Although effective stress analysis methods are considered by many practitioners to be superior, the majority of CIP concrete friction piles in the Winnipeg region are still designed using the α-method based on the results of undrained shear strength testing. Of interest is a comparison of the pile capacity that would be determined using the two methods for a typical soil profile and pile type/diameter. In this regard, each of the methods is described below using working stress design methods.

The total pile capacity (R) is made up of the summation of shear stress along the pile shaft (R_s) with a portion of the shaft (usually 1.5 m) neglected to account for installation and environmental effects (like shrinkage due to desiccation and freezing near the ground surface) and the total toe (base) resistance R_t. The general form of the equation is shown in Eq. 6.1.

$$R = \sum_{z=0}^{L} C r_s \Delta z + A_t r_t - W_p \qquad \text{Eq. 6-1}$$

where:
R = Total Pile Capacity

L = embedded pile length

z = depth

C = pile circumference

r_s = unit shaft resistance (adhesion) between the pile and soil

A_t = pile toe area

r_t = bearing capacity at toe

W_p = pile weight – often excluded from the determination of total pile capacity as it is very close to the weight of soil removed (there is little to no net increase).

Alpha Method to Determine Shaft Resistance

With the α-method, the ***unit shaft resistance*** of the pile is assumed to be proportional to the undrained shear strength of the clay as shown in Eq. 6.2. The undrained shear strength generally varies with depth and the ***ultimate total shaft resistance*** **(R_s)** of the pile is therefore the summation of the product of the adhesion and pile area for all of the soil layers under consideration.

$$r_s = \alpha\ s_u \qquad \text{Eq. 6-2}$$

where:
r_s = unit shaft resistance (adhesion) between the pile and soil

α = empirical adhesion coefficient (alpha)

s_u = undrained shear strength

Equation 6-2 relies on the determination of the empirical adhesion coefficient α which is essentially a proportionality coefficient, used to describe the portion of the measured undrained shear strength of the clay that is responsible for transferring the pile load (Q) into the soil mass surrounding the pile. It can also be considered as the ratio between the ultimate adhesion (c_a) to the ultimate cohesion along the pile shaft (c_u), that is c_a/c_u. Values of α have been empirically determined using back analysis from static pile load tests and have been found to be influenced by several factors including undrained shear strength (s_u), the stress history of the soil deposit, pile type (concrete or steel), the method of installation and the length of the pile.

There are several methods provided in the literature to determine an appropriate α value based on undrained shear strength. The Canadian Foundation Engineering Manual (CFEM) 4th Edition provides a graph showing the relationship between α and undrained shear strength but it is not indicated whether these results are for driven piles or drilled shafts. However, when the formula for the trend line included in the CFEM is superimposed on work by Terzaghi, Peck and Mesri (1996), it would appear that the formula would be applicable (perhaps more so) to drilled shafts as shown on Figure 6-3. By observation, the relationship shown on Figure 6-3 is highly variable and therefore of questionable reliability.

Based on Figure 6-3, an α value of 0.75 may be appropriate for determining the frictional resistance of a cast-in-place concrete pile embedded in Winnipeg clay with an average undrained shear strength of 50 kPa. However, in consideration of the decreasing overconsolidation ratio with depth in Winnipeg clays, determining average undrained shear strengths for discrete layers rather than determining an average strength along the entire pile shaft length may yield more accurate solutions. For WSD, a safety factor of 2.5 would generally be applied to determine an allowable unit shaft resistance of 15 kPa.

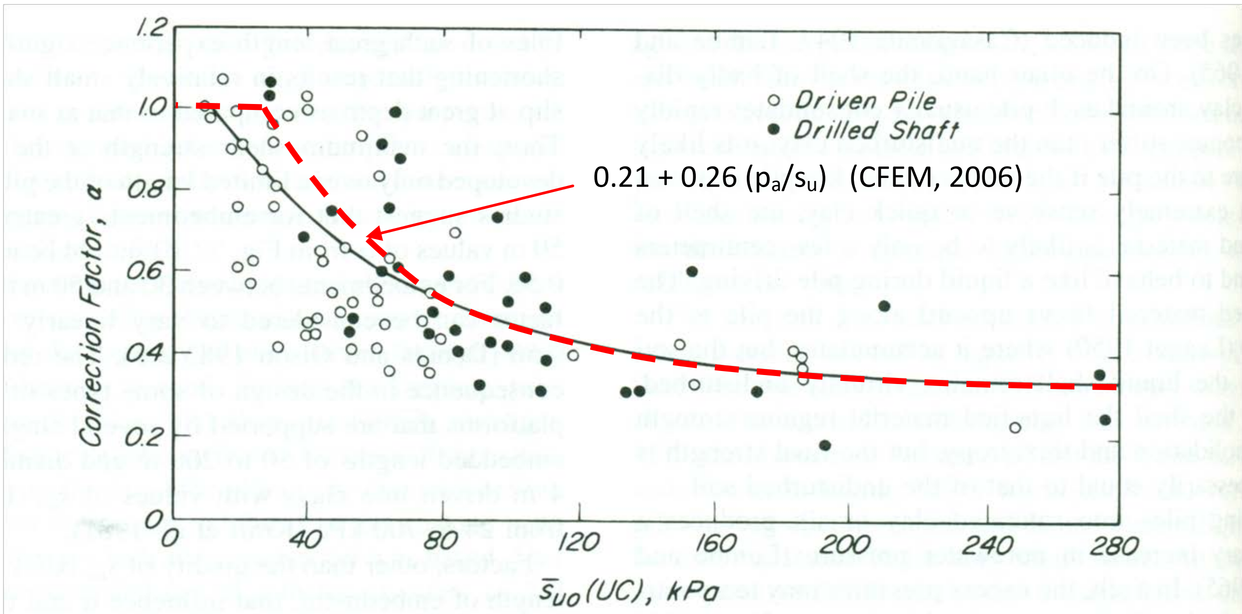

Figure 6-3 Adhesion factor as a function of undrained shear strength (modified from Terzaghi, Peck and Mesri 1993)

The question arises about how the combination of undrained shear strength s_u and α is assessed in Winnipeg practice for estimating the allowable shaft adhesion r_s. For WSD, some practitioners use one-half of the unconfined compression strength q_u for the shaft friction term r_s, although often with a FS of 3 rather than 2.5. This essentially assumes that α is 1.0 which, based on Figure 6-3, is rather high for the typical shear strengths in Winnipeg of about 40 – 60 kPa, or alternatively, assumes that the slippage occurs through the soil rather than the pile-soil interface. In this regard, investigations by Yaipukdee (1968) suggests that the strength increases with increasing distance from the pile and that at a distance of about 6 mm, is equal to the intact shear strength of the soil. It is known, however, that at low to moderate stress levels, $q_u/2$ is usually a low estimate of s_u. In practice, a combination of a low s_u and a high α appears to give a usable estimate of pile adhesion. Following this design philosophy, the allowable unit shaft resistance (for the 50 kPa soil in the previous paragraph) would be 17 kPa (compared with 15 kPa). The author believes that this approach has been calibrated specifically to the Lake Agassiz clay in Winnipeg and should not be used elsewhere without careful consideration.

Beta Method to Determine Shaft Resistance

There is also a second form of analysis known as the β-method, which is based on effective stress analysis (Burland 1973, CFEM 2006). Here, r_s (or q_s in CFEM) = $K_s \tan \delta\sigma'_z = \beta\sigma'_z$. This method is considered state-of-practice in much of Canada but is not widely used in Winnipeg. As with the undrained analysis, there are several different methods to determine the term β. The CFEM provides a range of 0.25 to 0.32 for β in clay. Other methods take into consideration the overconsolidation ratio of the clay with suggested corrections for drilled shafts. Burland (1973) suggests $\beta = (1-\sin \varphi') \tan \varphi'$ which would result in a value of 0.22 for a typical effective stress friction angle of 17 degrees for Winnipeg clay. Meyerhof (1976) suggests using the relationship shown in Eq. 6-3.

$$\beta = K_s \tan \varphi'_a \qquad \text{Eq. 6-3}$$

where: $K_s = K_o = (1-\sin\varphi')\sqrt{OCR}$

and φ'_a can be taken as φ'

For example, β would equal 0.31 and 0.43 for OCRs of 2 and 4 respectively assuming that $\varphi' = 17$ deg. Based on an assumed OCR profile ranging from 5.5 to 2 and a 10 m pile length, the unit shaft resistance ranges from 4 kPa (at 0.5 m) to 31 kPa at 9.5 m using the Beta method. Again, assuming a safety factor of 2.5, the average (allowable) unit shaft resistance would be 10 kPa. This value is about half of the capacity which would be determined using the alpha method but shows good agreement with values reported by

Spencer (1982). In it, the results of a pile load test in north Winnipeg yielded an ultimate unit adhesion value ranging from 22 to 23 kPa. For small displacements (2.5 to 5 mm) and a safety factor of 2.0, the allowable adhesion value was reported to be 10.8 to 11.5 kPa (Kjartanson 1983). In this field test, a void space was provided at the base of the test pile to eliminate end bearing support.

Toe Resistance

The ***ultimate toe (end bearing) resistance*** of a single pile (R_t) can be taken as:

$$R_t = N_t\, s_u\, A_t \qquad \text{Eq. 6-2}$$

where: R_t = Ultimate Toe Resistance at Base of Pile

N_t = Bearing Capacity Factor (ranges from 6 to 9, depending on pile dia.)

s_u = Undrained shear strength

A_t = Pile Tip End Area

In local practice, end bearing is often omitted in the determination of total pile capacity for a CIP concrete friction piles in clay; this is thought to be due to the potential for soil disturbance or sloughed material at the bottom of the drilled shaft. Unfortunately, there is only very limited load test information for cast-in-place concrete friction piles in Winnipeg that would provide information relative to the load sharing between shaft resistance and end bearing and the α-value applicable to local soil conditions and installation methods. To date, most local practitioners have accepted the use of an adhesion factor (α) and exclusion of end bearing based on the common knowledge that very few performance related problems have been reported and that the contribution of end bearing is relatively small compared with the shaft resistance (Figure 6-4). A later paragraph will show that at working loads, only the upper part of the pile is transferring load to the soil and little to none of the load reaches the bottom of the pile. The assumption of a constant working stress along the full length of the pile is therefore probably simplistic.

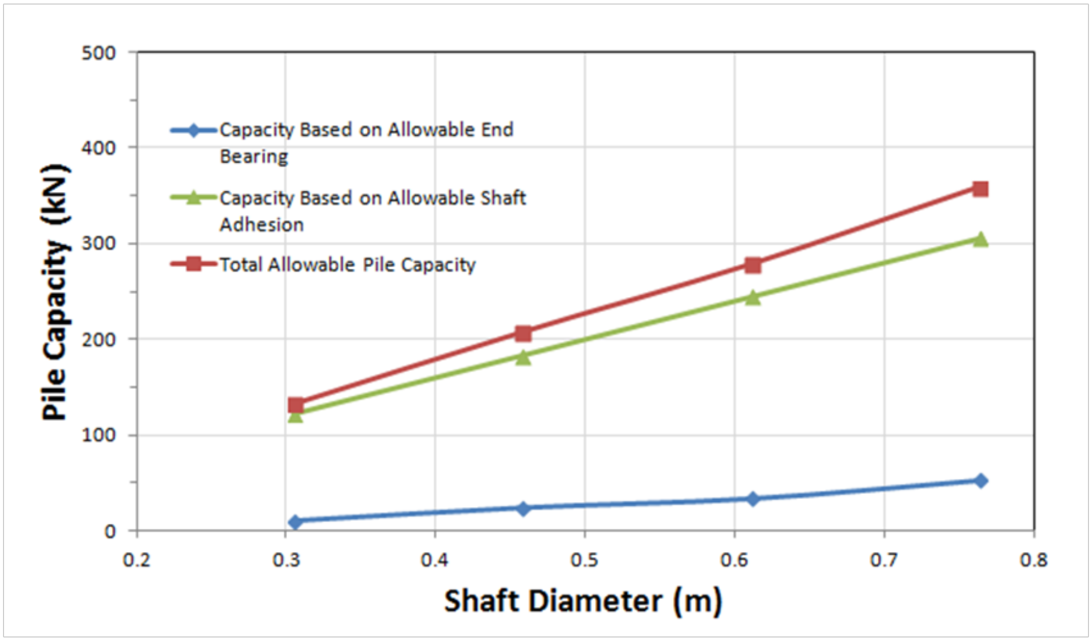

Figure 6-4 Relationship between allowable shaft resistance and allowable end bearing for 10 m long drilled shaft, based on an undrained shear strength of 50 kPa, $\alpha = 0.75$, FS shaft = 2.5, FS base = 3.0 and the upper 1.5 m neglected from capacity determination

It has become apparent in some cases that the introduction of LSD in local practice has led to a need for additional CIP concrete friction piles compared to designs based on WSD, even without any change in loading conditions. If the historical approach has proven successful, it is not surprising that questions are being asked if the LSD design approach has added unnecessary conservatism. The answer requires an understanding of the interaction between the soil and the pile.

Full mobilization of shaft resistance and base resistance depends on the axial displacement of the pile under loading. In this regard, it is important to recognize that firstly, the manner in which adhesion is mobilized depends on the soil type; for fine grained soils, the load transfer is non-linear and decreases with depth. In such a case, the elastic compression of the pile is greater at the top portion of the pile than at the bottom and allows more efficient load transfer from the pile to the surrounding soil. Secondly, the full shaft and end bearing capacities are not mobilized at the same amount of displacement; skin friction is fully mobilized before any appreciable base resistance is developed and may occur at vertical displacements of only 0.5 to 1% of the pile diameter (2 to 5 mm for a 457 mm diameter pile). Full mobilization of base resistance requires much greater displacement, perhaps as much as 20% of the pile diameter (91 mm for a 457 mm diameter pile). From a practical perspective, this number is often taken as 5% of the shaft diameter (23 mm for a 457 mm diameter pile).

If it is assumed that the settlement associated with the allowable capacity determined using WSD (15 kPa) is tolerable, and that a safety factor of 2.5 is appropriate, the nominal unit shaft resistance would be 37.5 kPa. Based on Ø = 0.4, the factored ULS unit shaft resistance would therefore also be 15 kPa which in this example may be the limiting case for design. If however, base capacity is considered, a factored end bearing resistance of 180 kPa would be added (also based on a resistance factor of 0.4). For example, the factored ULS capacity with and without end bearing for a 450 mm diameter by 10 m long friction pile is 180 and 210 kN respectively. The exclusion of end bearing therefore results in a reduction in total ULS pile capacity of about 14%.

The inclusion of end bearing can be justified for the ULS since the pile must fail in plunging and hence end bearing resistance must occur leading to failure. Inclusion of end bearing for the determination of the SLS however should generally not be considered as it is likely only to develop fully at displacements greater than what would be considered acceptable. The preceding paragraphs highlight the importance of carrying out pile load tests to assist in the implementation of Limit States Design methods using load settlement data based on local soil conditions, and hence pile behaviour.

6.2.6 Driven Friction Piles

Timber piles have been successfully used in the Winnipeg area for many years to support light to moderately loaded structures. Although not commonly used in Winnipeg, pre-cast concrete and steel friction piles can also be used for lightly loaded structures (these piles are typically driven to practical refusal onto a hard stratum). The procedures used to determine α and β-values for driven friction piles depend on factors that include the pile type (material) and soil type.

A summary of adhesion factors (α) in clays from various sources is shown on Figure 6-5 (McClelland, 1974). For comparison, the relationship presented in the Canadian Foundation Engineering Manual (2006) has been added to Figure 6-5. For example, Winnipeg clay with an undrained shear strength of 50 kPa would have an adhesion value ranging from about 0.7 to 0.9. The reduction in α at higher undrained shear strengths is believed to be due to the effects of "whipping" of the pile during driving and enlargement of the aperture within the soil. It may also be due to the level of disturbance associated with driving piles into stronger, possibly overconsolidated clay and the reduction in shearing resistance from "peak" to "post-peak" strength.

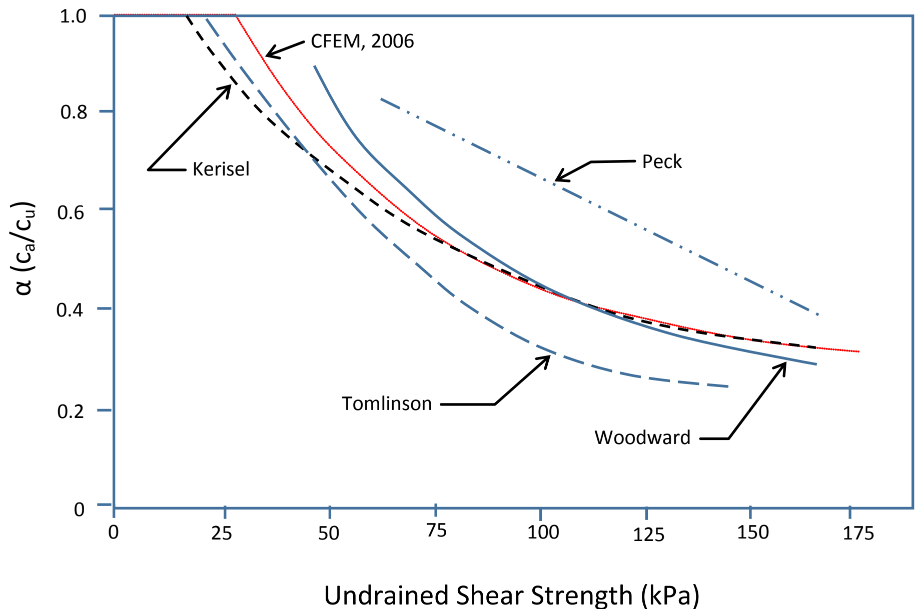

Figure 6-5 Relationship between adhesion factor (c_a/c_u) and undrained shear strength for driven piles (modified from McClelland 1974). With permission from ASCE

6.2.7 Cast-in-Place Concrete Caissons

Cast-in-place concrete piles (caissons) ending in dense till may be either straight shaft or have an expanded (belled) base. Typical allowable bearing capacities derived from WSD are 718 to 957 kPa (15 - 20 ksf) for competent till and as high as 1436 kPa (30 ksf) for very dense till. However, these capacities require that hand cleaning is possible at the base of the excavated shaft, a practice which is becoming less common. If hand cleaning is not possible, the bearing capacity of mechanically cleaned bases is typically reduced by 1/3. The selection of the appropriate end bearing value is based on interpretation of subsurface conditions, in particular the compactness condition and water content of the till. Often, the capacity of the shaft through the clay is neglected in determining the total pile capacity. The following paragraphs look more closely at the implications of this and the importance of the condition of the base of the caisson.

Figure 6-6 illustrates the differences between allowable capacities of straight shaft and belled caissons bearing on a very dense till where the bell is about 2.5 times[21] the shaft diameter. The pile length is assumed to be 15 m and the upper 1.5 m of the pile is excluded in the determination of ultimate shaft capacity. The allowable bearing capacity of the till is assumed to be 480 kPa, representative of a competent till with a mechanically cleaned base. The allowable adhesion in the clay is assumed to be 15 kPa based on an α-value of 0.75 and safety factor of 2.5.

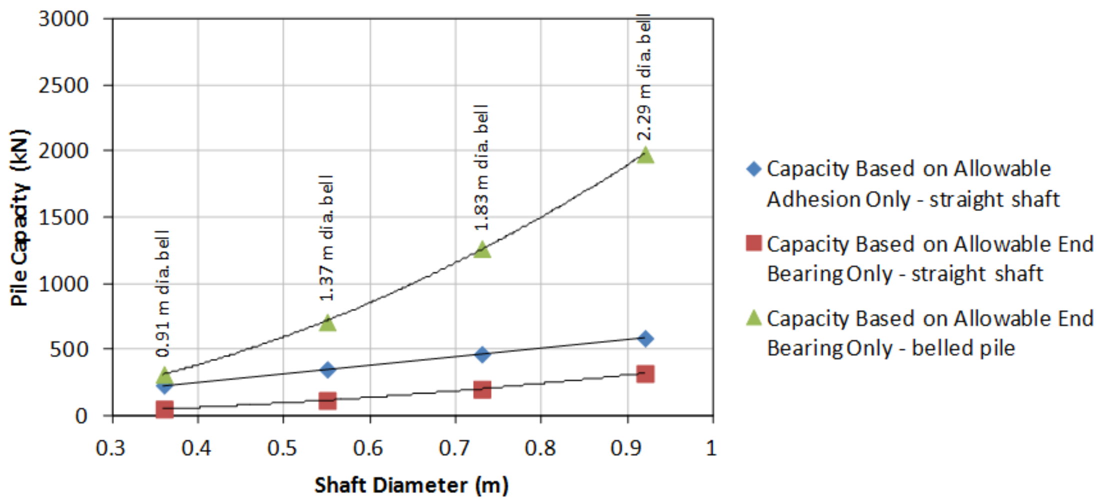

Figure 6-6 Relationship between allowable shaft resistance and allowable end bearing for straight shaft and belled piles with **mechanically cleaned base**

For a straight shaft, the allowable shaft capacity is larger than the allowable base capacity, with the ratio decreasing as the pile diameter increases. It does so because the end bearing capacity increases with the square of the diameter, while the shaft capacity increases linearly. As the length of the caisson increases, neglecting shaft capacity may result in an overly-conservative design. For a belled pile, the end bearing capacity is

[21] This ratio may be as high as 3 but locally, 2.5 is more typical

considerably higher than the shaft capacity and increases more rapidly than the shaft capacity throughout the range of diameters considered. In this case, it may be more reasonable (or less conservative) to neglect shaft resistance in the clay given the base capacity offered by the much stronger till unit. Note however, that larger settlements are needed to mobilize the ultimate base capacity of larger belled piles and this influences values that can be used for determining allowable capacity.

Only small settlements are required to fully mobilize shaft friction, a loading condition where the SF for the shaft essentially becomes 1.0. Corresponding to these small settlements, the base of a belled pile will still be a long way from mobilizing its ultimate bearing capacity and will therefore be operating at a SF much greater than 1.0. As seen on Figure 6-7, the ultimate shaft capacity and allowable end bearing capacity are equal at a pile capacity of about 1000 kN (where the lines cross). Furthermore, for a much shorter pile, the shaft adhesion would be relatively small in comparison with end bearing capacity, that is, the lines would cross at a lower pile capacity value.

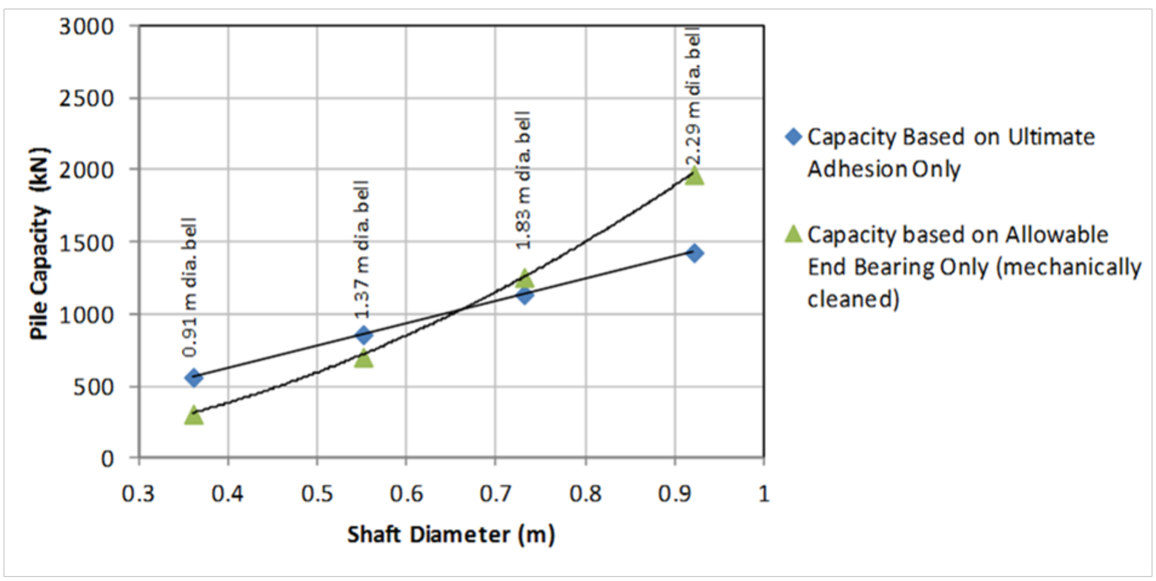

Figure 6-7 Relationship between ultimate shaft resistance and allowable end bearing for belled piles with mechanically cleaned base

For the example in Figure 6-7, it may be appropriate to use a higher allowable end bearing capacity of 718 kPa, but only to a maximum pile capacity of about 1000 kN. For loads higher than 1000 kN, it would be advisable to use a reduced allowable end bearing capacity since any additional applied load is transferred directly to the base. This is believed to be the justification for the 1/3 capacity reduction employed in local practice for a mechanically cleaned base. This point can be emphasized further if the allowable end bearing capacity for a hand cleaned caisson is compared against the ultimate adhesion value in Figure 6-8. In this case, the ultimate shaft capacity is equal to the allowable end bearing capacity at about 700 kN. Clearly, hand cleaning (if permitted) or very thorough mechanical cleaning is advisable if the maximum allowable end bearing is used to determine pile capacities.

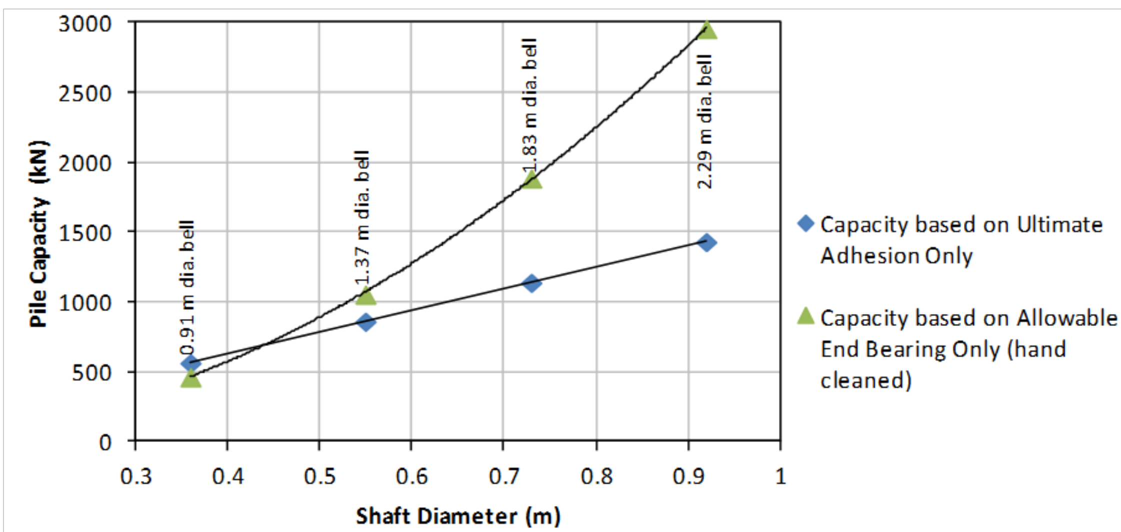

Figure 6-8 Relationship between ultimate shaft resistance and allowable end bearing for belled piles with **hand cleaned base**

With respect to Limit States Design, historical capacities and local experience should at least be considered in determining ULS and SLS capacities. In this regard, the ULS capacity should be based on both nominal shaft resistance (based on undrained shear strengths and an appropriate α-value) and end bearing capacity, with both of these values reduced using appropriate resistance factors (φ). The Author recommends that the use of nominal capacities higher than what would be derived from WSD methods should be based on measured values or load test results. Clearly, there may be a significant advantage in carrying out such tests that may allow the use of higher nominal values in the determination of the ULS capacities.

For settlement-sensitive structures, the Author recommends that capacities for the Serviceability Limit State be based on detailed settlement analysis using empirical parameters, or preferably based on the results of static load testing. Based on the successful performance of structures supported by caissons in Winnipeg, the determination of the SLS by applying a conventional safety factor of 3 to the ultimate (nominal) bearing capacity used to determine the ULS capacity may be otherwise justified.

6.2.8 Rock-socketted Caissons

Rock-socketted caissons develop their capacity through circumferential bonding at the shaft perimeter between the concrete and rock, and on end bearing on sound rock. The sharing of load between the shaft (socket) and base is largely dependent on the length of the socket relative to its diameter and to some degree, the ratio of the elastic modulus for the concrete and limestone (Tomlinson 2008). It is generally accepted that when the length to diameter ratio is greater than about 4, the contribution to total capacity from end bearing is small or negligible. Common local practice is to design rock-socketted caissons in sound bedrock using an allowable circumferential bond strength between the concrete and rock of 1030 kPa (150 psi) plus an allowable end bearing support of 2870 kPa (60 ksf). The bond value is based on a percentage of the compressive strength of concrete, assumed to be about 35 MPa. Reduction factors are often applied for fractures or soft rock and end bearing may only be included if proof cores can bet taken from the base of the pile to confirm intact rock exists below the base. There has been limited experience locally using Limit States Design methods for the design of rock-socketted caissons. However, the NBCC resistance factors in Table 6-3 would still apply.

The inclusion of end bearing generally requires that the competency of the bedrock at least two pile diameters below the base is confirmed. Until very recently, this was often accomplished by drilling a small diameter hole at the base and using a feeler gauge to detect open fractures. If an open fracture is encountered, the shaft may need to be extended until competent rock can be confirmed. However, this procedure requires worker access into the hole for drilling, probing and inspection - a task which is becoming increasingly uncommon. Exclusion of the proof core requires that either end bearing be neglected, or alternatively a test hole is drilled from ground surface with bedrock coring to the target depth (say 3 m below the design base elevation) to confirm bedrock competency at each foundation unit. The cost of the drilling may be easily offset by reducing the length of the socket (excluding end bearing) that would otherwise be required.

There would be a considerable benefit to local geotechnical practice to carry out load tests on a variety of drilled shafts founded in the clay, till and in bedrock. The results of static load tests would provide valuable load settlement-curves for determining the nominal resistance at acceptable displacements of the pile. Alternatively, O-cell tests would provide the same information but with the ability to separate the measured tip and shaft resistance components. The author is aware of recent work done in this regard for the design of rock-socketted caissons in downtown Winnipeg but the results have not yet been published.

7 RIVERBANK STABILITY

7.1 HISTORICAL PERSPECTIVE

Winnipeg is situated at the confluence of the Red and Assiniboine Rivers at the site of Aboriginal settlements dating back many hundreds of years. The first written documentation of the site was made by Pierre de La Verendrye in 1738 while searching for a water route across the continent to the Pacific Ocean. There is speculation that the site may have been used as long ago as 3,500 years by the forefathers of Canada's native people (Armstrong 1986). River lots and established fur trade routes shaped the city in its early years during a time where the rivers and streams were the young city's main "roadways" both in the summer and winter. By 1875 however, many homes and businesses began fronting on the newly developed streets and turned their backs on the river. At the same time, Winnipeg's central geographical location made it a natural hub for the expansion of the transcontinental railway, beginning with the completion of the Canadian Pacific Railway in 1885. It is somewhat ironic that it was the development of the railways that led to the neglect of much of Winnipeg's downtown riverfront properties.

Throughout most of the 20th century, the commercial uses of the waterways that were central to the life of the city dropped off as a consequence of development and many stretches of older riverbanks within the city became a dumping ground or in extreme cases, industrial wastelands. Fortunately, in the 1980s, the City of Winnipeg began turning its attention towards the riverbanks with notable enhancements that now include the Forks Development, the Canadian Museum For Human Rights, Shaw Park, and areas of dedicated green space to name a few. The author has clear recollections of the former condition of the area of the Forks Development which formerly served as industrial sites and rail yards with little to no provision for the public to access (Figure 7-1). Anyone familiar with the current site may agree that the transformation has been impressive. Local geotechnical practitioners will be familiar with the slope stability considerations associated with the development.

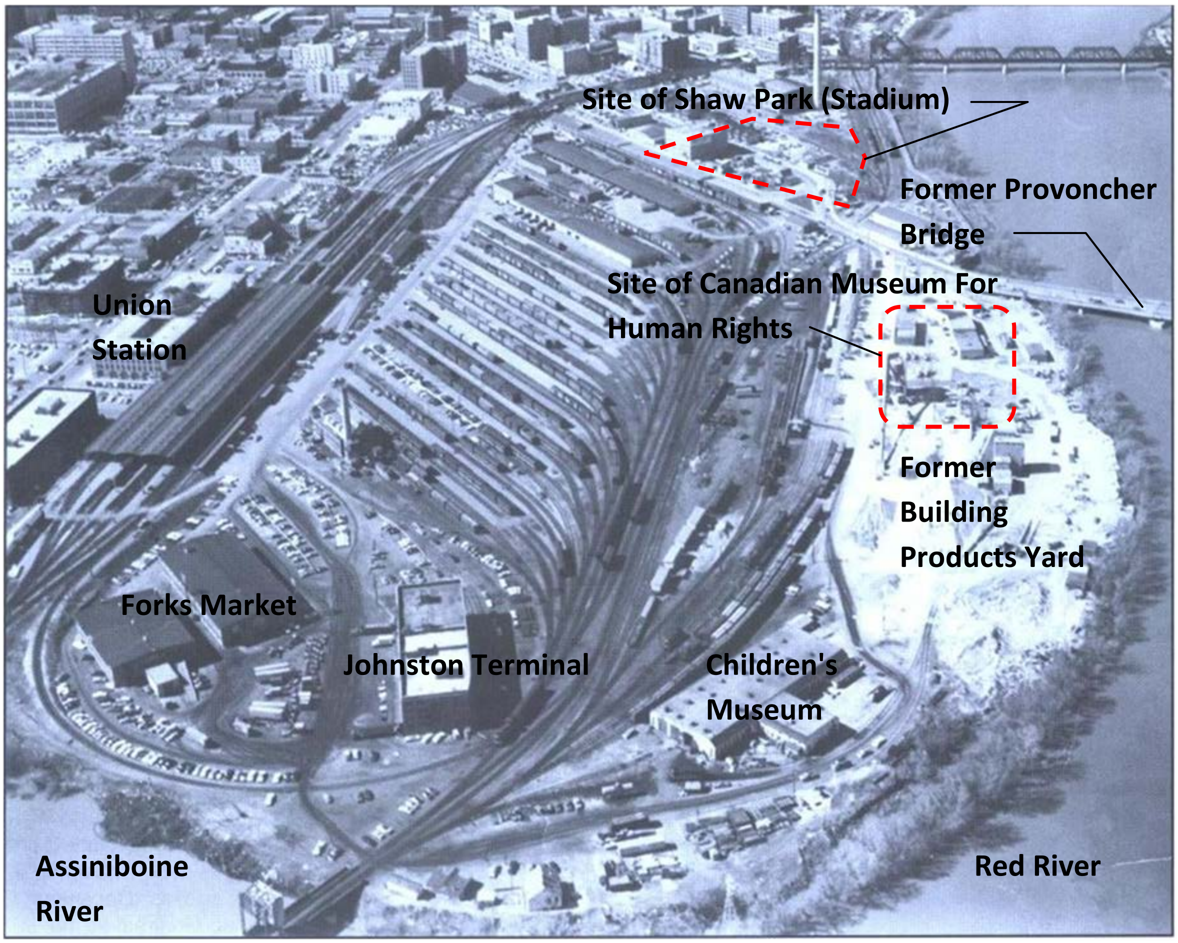

Figure 7-1 East Yard at the Site of the Forks Development, c.1970
(University of Manitoba Archives, Winnipeg Tribune fonds, PC-18-1364-011)

Unfortunately, public access to many of the waterways is fragmented and in many cases, access to linear features along the riverbanks is not possible without trespassing on private property. Although community organizations and the City of Winnipeg often collaborate on measures to expand, protect and enhance natural riverbanks throughout the city, there are also opposing interests such as commercial developments that sequester riverbank properties as they become available. While we cannot change history, we should take advantage of opportunities to enhance our waterways and provide access to what perhaps should always have been a public amenity.

7.2 THE CITY'S WATERWAYS

Waterways within the City of Winnipeg include the Red River and its tributary rivers, creeks and channels. The locations of these waterways are shown on Figure 7-2. In decreasing size, the main tributaries are the Assiniboine River, La Salle River, Seine River, Sturgeon Creek, Omand's Creek, and Truro Creek. Smaller tributaries to the Red River include Bunn’s Creek in East Kildonan and the Cordite Ditch which drains into it. All of these waterways (with the exception of the Cordite ditch) fall under the jurisdiction of the City of Winnipeg's Waterways Section. There are also five unnamed creeks near Headingley and Beaverdam Creek in Charleswood which are tributaries to the Assiniboine River. Although it is an extension of the Red River, the Floodway channel (shown as a blue dashed line on Figure 7-2) is not generally considered to be a waterway because of its intermittent use.

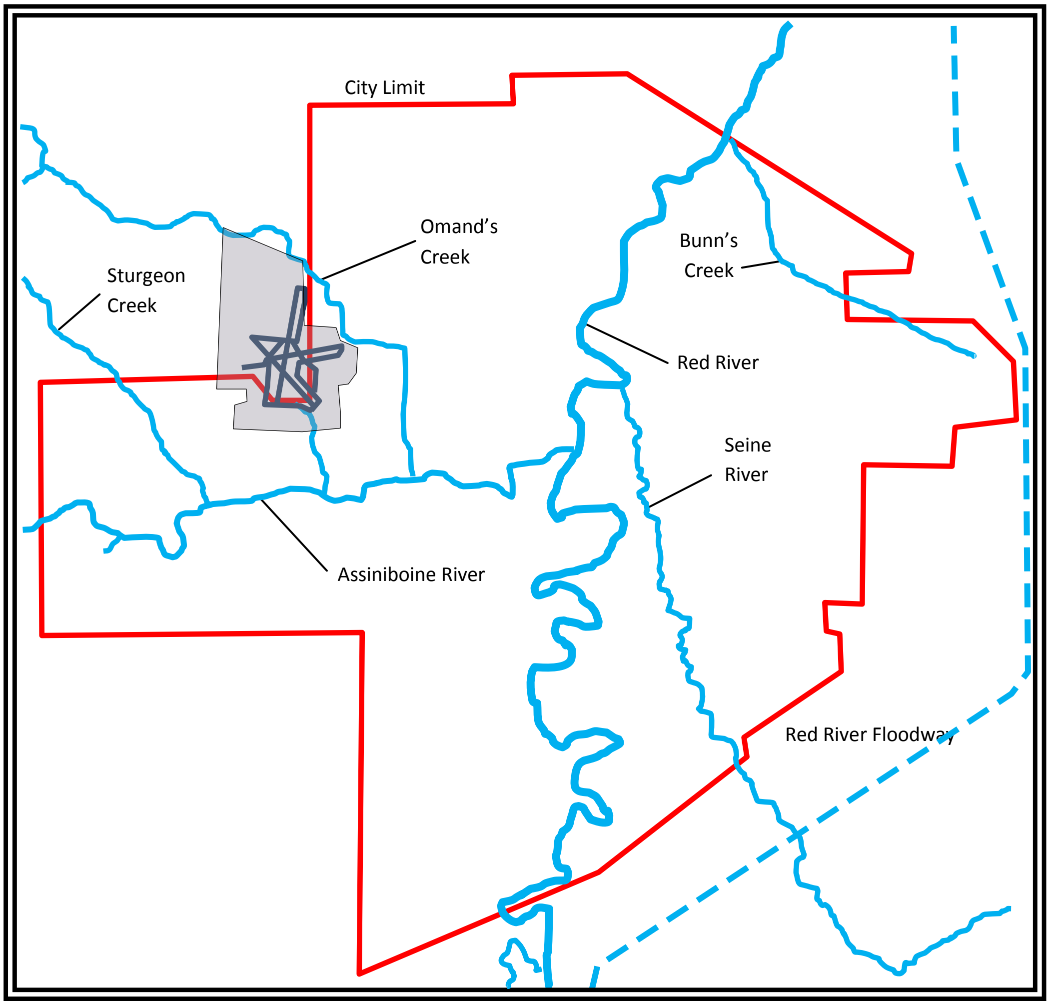

Figure 7-2 Location of waterways within Winnipeg

7.3 RIVER MORPHOLOGY AND CHARACTERIZATION

All four rivers within the city have meanders and relatively straight sections, most notably along the Red River which flows northward into Lake Winnipeg. From the top of bank to the bottom of the channel, the riverbanks measure from 9 to 15 metres, with the highest bank sections along the Red River. Along some stretches, in particular in the western and extreme northerly sections of the city, channel down-cutting is limited by the presence of till and bedrock. An example is upstream of Assiniboine Park where the Assiniboine River channel bed is lined with coarse material from the till matrix. In stretches where the till surface is relatively deep, such as the downtown area of Winnipeg, deep river thalwegs are common where the channel has down-cut into the clay. Often these thalwegs are infilled with sandy alluvium, depending on channel velocities. Because of this, cross sections of river bottoms used for slope stability analysis can vary from year to year.

Typical of meandering rivers, slow changes in the river courses through Winnipeg have occurred in recent geologic time. The migration of meander loops has resulted in the erosion of lacustrine banks (clay dominated) on outside bends and the deposition of alluvial soils (silt and sand dominated) on inside bends. Often the alluvium is laid down over lacustrine soils below the depth of river down-cutting. These layers of lacustrine soils may be thin but still may significantly impact bank stability because of their comparatively low shear strength. The progression of meander loops is often accelerated by bank erosion and ice scouring during spring floods.

The City of Winnipeg Waterways Section has routinely taken high resolution aerial photographs of the waterways, with the most recent photography in the fall of 2013. The two most recent set of aerial photos (2008 and 2013) have been orthorectified (geo-referenced to the city base map) using a digital elevation model (DEM). This allows single mosaic files to be created. The DEM from 2008 was used to create a contour plan that can be used to aid in the characterization of riverbanks, in particular the identification of alluvial and lacustrine banks. For example, a stretch of the Red River in the vicinity of the Kildonan Settlers Bridge and the former CPR "Bergen Cut-Off" bridge is shown on the 2013 orthophoto in Figure 7-3 with 1998 contours (at 0.25 m intervals) superimposed.

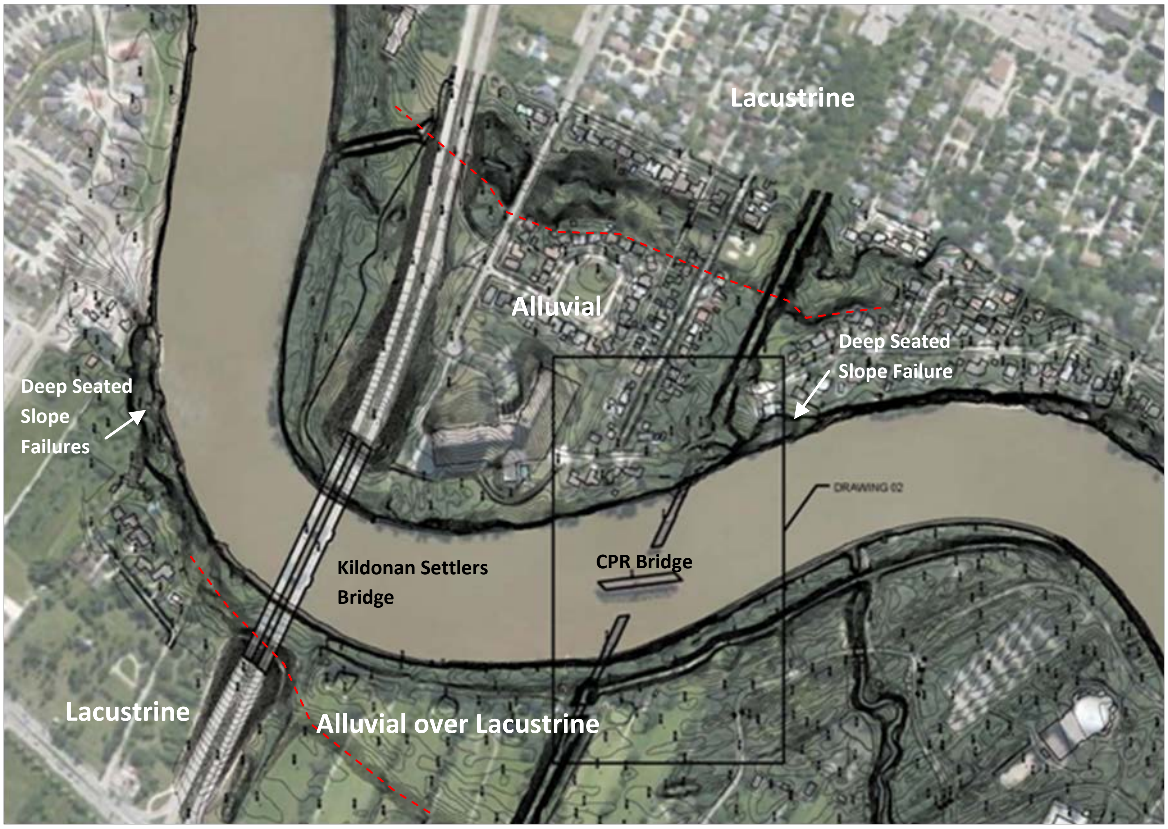

Figure 7-3 Orthophoto with contours (from City of Winnipeg Waterways Section)

This stretch of the river contains several unique features inside the red-dashed boundaries, including significant areas of alluvial deposits on the inside (east) bend of the meander with lacustrine and alluvial-over-lacustrine deposits on the outside bend. The alluvial banks are generally lower in elevation than lacustrine banks which are undergoing erosion for the first time since glaciation. Several natural features can be used for determining such boundaries. These include the topography on the east side, which has a low terrace defined by contour lines some distance from the river that denote the limit of alluvial deposits. The elevation of the top of the west bank is higher and shows evidence of arcuate failures that are typical of lacustrine deposits.

A Lidar survey of the City of Winnipeg was undertaken in 2011 by the Water and Waste Department. This can also be used to aid in the characterization of riverbanks, in particular when viewed using various display characteristics available in GIS data processing applications. Examples of the Lidar image from the area of the Kildonan Settlers Bridge are shown in Figure 7-4. Using both colour and black and white elevation gradients, even very subtle changes in ground elevation are evident.

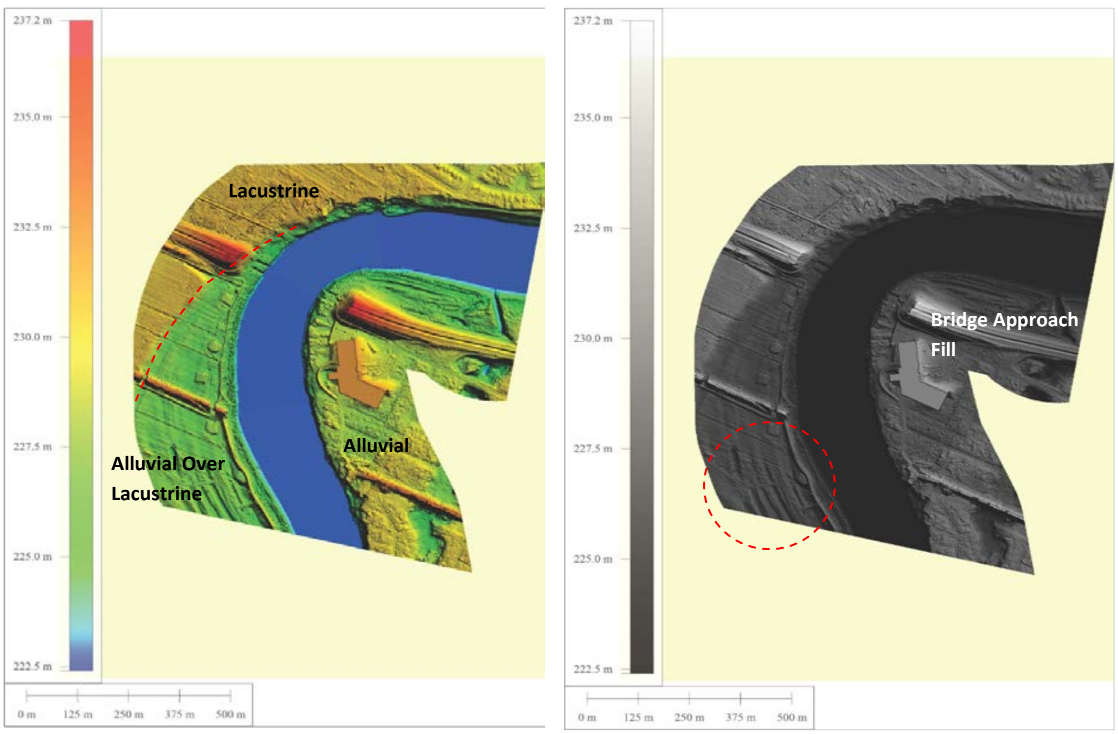

Figure 7-4 Lidar images in vicinity of Kildonan Settlers Bridge (data file courtesy of City of Winnipeg Water and Waste Department)

Both images clearly show the boundary between lacustrine banks where deep seated rotational failures are evident, and areas of alluvial soils. Of particular interest are the ridges of alluvium evident on the west side of the channel in the flood-prone section of the Kildonan Golf Course which is now protected by an earth dike (visible within the dashed red circle on the right hand image).

GIS data processing applications such as Global Mapper® are capable of quickly generating profiles across the river channel. Figure 7-4 shows an example of a profile cut across the channel downstream of the Kildonan Settlers Bridge. The difference in elevation on the lacustrine bank (shown as "L" on Figure 7-5) on the north side of the river (about Elev. 230.5 m) is about 2.5 metres higher than the alluvial ("A") bank on the south side of the river. The slump block feature associated with the failure on the west side of the river is also evident.

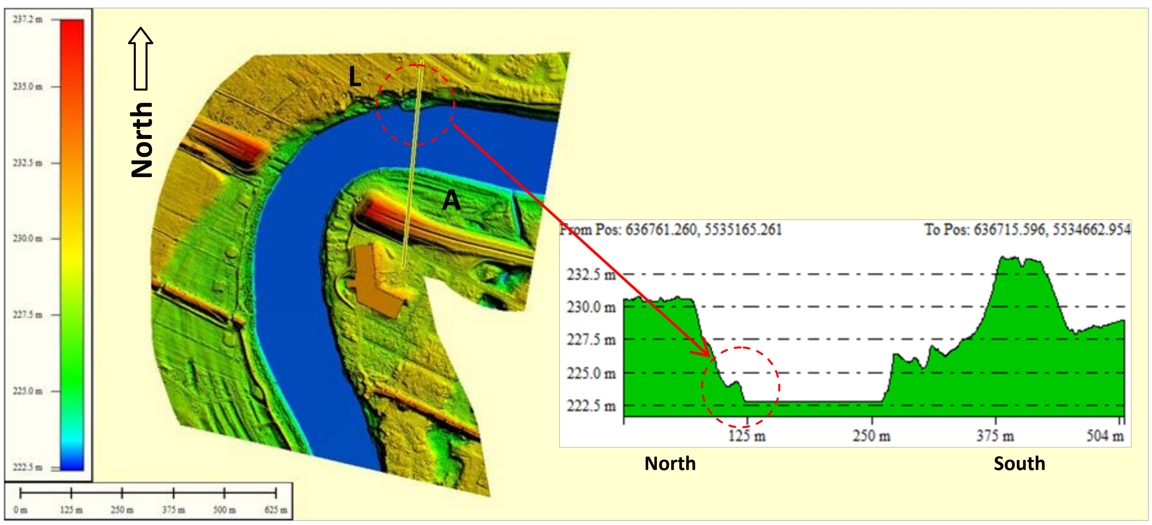

Figure 7-5 Profile across river channel using Lidar data (data file Courtesy of City of Winnipeg Water and Waste Department)

Another useful feature is the ability to view the Lidar data in 3D, allowing the site to be viewed in 360 degree orientations with any of the display characteristics available in the 2D (plan) view. One example is shown in Figure 7-6 that shows the river channel viewed from the west side (facing east). The failures on the outside of the bend downstream of the bridge can be clearly seen.

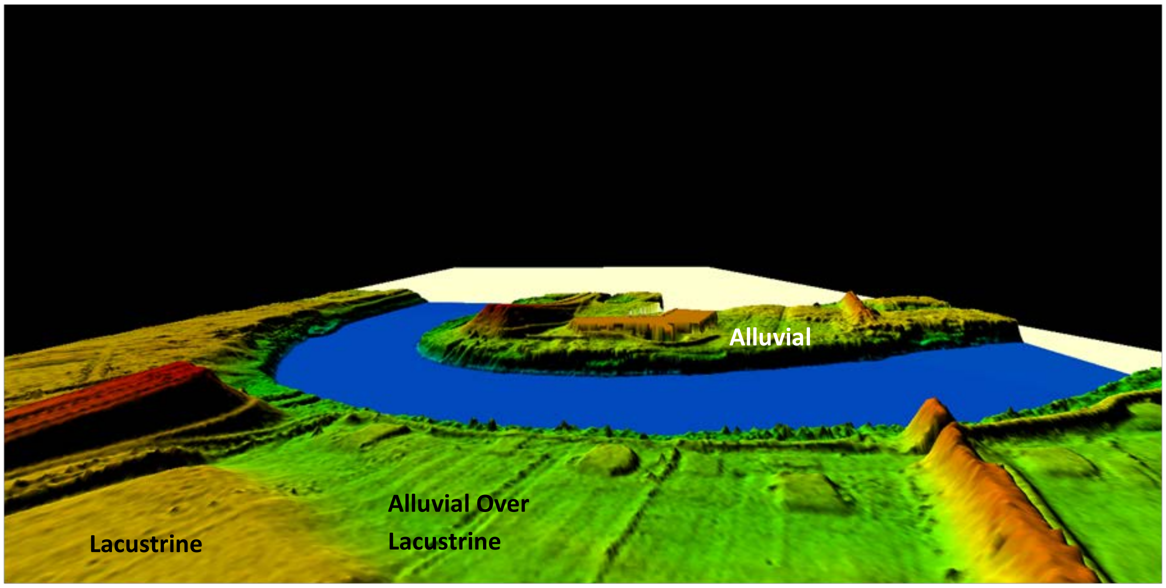

Figure 7-6 3D Perspective

7.4 SLOPE STABILITY ANALYSIS

7.4.1 Historical Review

The Greater Winnipeg Dyking Board and the National Research Council investigated riverbank failures following the 1950 flood and concluded that under the post-flood drawdown conditions, the averaged cohesive strength of the clay was about 600 psf (Baracos 1950). Baracos (1953) concluded that although laboratory tests from borings at sixteen locations along the proposed Floodway route showed undrained shear strengths much higher than 600 psf, it was considered prudent to use the lower bound (600 psf) along circular slide planes as determined from the previous studies along the Red River banks for the design of the Floodway slopes.

In 1960, a survey of the riverbanks along the Assiniboine River and Red River was undertaken to determine the slope at which natural banks in clay failed. At the urging of Professor A. Casagrande, a graphical relationship between total bank height and steepness as shown on Figure 7-7 was generated (Mishtak 1964). This work showed that over 80% of the outside bends of the river (in mainly glaciolacustrine clay) became stable at slopes ranging from 4.5H:1V to 6.75H:1V. This key observation became the basis for selecting 6H:1V sideslopes for the Floodway channel. While it was recognized that some failures might still occur, it was concluded that it would be more economical to repair problem areas as opposed to flattening the sideslopes further. When asked to comment on the stability of Winnipeg riverbanks, Professor Casagrande estimated that for a slope to be considered entirely safe, it should not be steeper than 1 in 6 if it has not experience sliding and 1 in 9 if it has been subjected to substantial sliding.

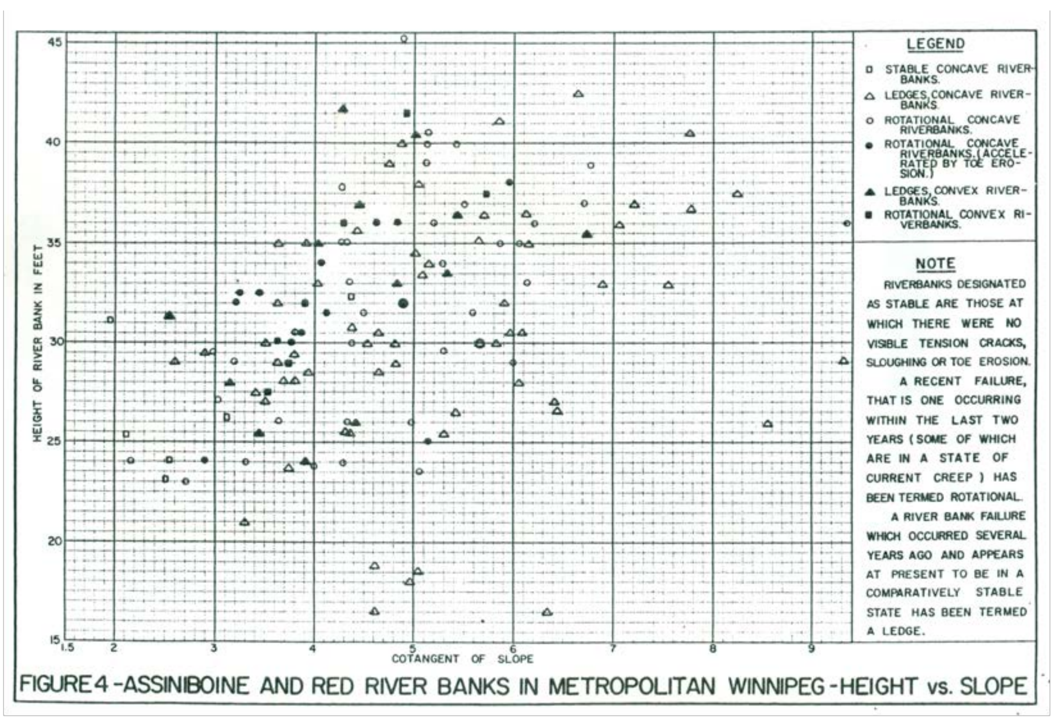

Figure 7-7 Slope height vs. cotangent of slope (Mishtak, 1964)

Following Professor Casagrande's recommendations (and at the same time as Mishtak's work was underway), the Soil Mechanics Division of the PFRA and the Government of Canada carried out extensive triaxial tests on Shelby tube and large block samples obtained along the proposed Floodway alignment. Other important work carried out included an instrumented test excavation (Anderson, Kenyon, Blatz, and Smith 2004). Both total and effective stress analyses were carried out based on the results of the test excavation assuming that failure surfaces were circular and their depth limited by the depth to till.

The total stress analyses indicated average undrained shear strengths of about 620 psf were required to achieve a safety factor (FS) of 1.0 (Mishtak 1964). The effective stress analysis was complicated by difficulty in modeling pore water pressures measured at the time of failure and pressure relief occurring in the till as a result of hydraulic fracturing at the base of the excavation. Depending on the shear strengths used for the analysis, factors of safety ranging from 1.28 to 2.08 were calculated for failure surfaces in the clay and as low as 0.65 for assumed failure surfaces passing through the non-cemented till. The results of the effective stress analysis for theoretical failure surfaces through the clay with a FS of 1.0 are shown on Figure 7-8.

Although the merits of effective stress analysis were recognized, the results were considered to be erratic and inconclusive, and emphasis was given to an observational approach of riverbank monitoring. In the concluding remarks in the 1964 report on the Preliminary Soil Mechanics Aspects of the Red River Floodway, Mishtak wrote *"The Floodway, although it has often been simply described as being a "big ditch", is largely dependent on the integrated application of several sciences. In this case, the science* *under consideration, that of soil mechanics, plays a predominant role in the success of the Red River Floodway. Its major function in selecting slopes for the excavation has involved extensive theoretical studies based on practical observations and data. Other factors under study also, cannot be overlooked, however, although extensive investigations and studies have already been conducted and more investigations are planned and some currently being conducted, the success of these investigations and studies will be evident only in the completed project itself".*

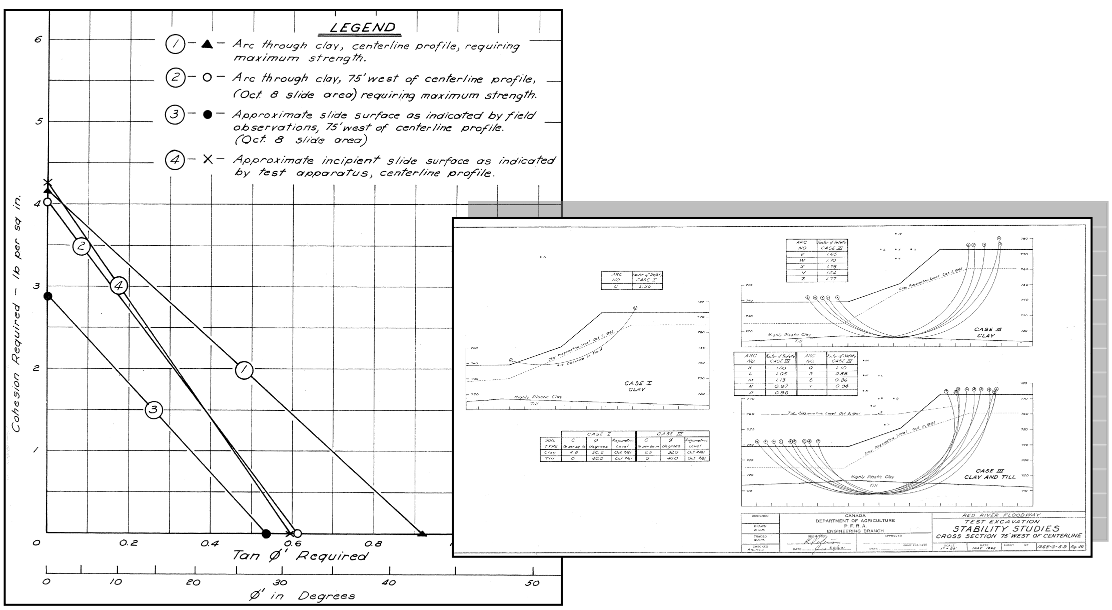

Figure 7-8 Results of total stress stability analysis (Mishtak 1964)

In a 1966 report to the Winnipeg Rivers and Streams Authority No. 1, Professor Hugh Sutherland concluded that the "*present methods of analysis are inadequate in themselves, and that these methods, while giving some guidance, must be tempered with substantial engineering judgement, supplemented by awareness and experience of the conditions in Winnipeg*" (Sutherland 1966). Professor Sutherland was not being critical of local practitioners but rather, simply commenting that they were being asked to analyze a problem for which there was an incomplete understanding. Professor Sutherland reported that "*the analysis of failed slopes have shown that the total stress (Ø = 0) and effective stress methods based on laboratory determined shear strength values, substantially overestimates the factor of safety of riverbanks*". He concluded that the actual shear strength of the clay at failure was considerably less than the measured (laboratory) values. This was the same conclusion drawn by Professor Baracos in his 1952 work.

Professor Sutherland recommended that a $\phi = 0$ stability analysis should be used for riverbank stability analysis provided that an undrained shear strength of 500 psf is used and a minimum safety factor of 1.5 is obtained. He rationalized that an analysis based on a limiting shear strength (500 psf) would allow factors such as bank height, slope geometry and variation of river level to be taken into account. Interestingly, he also concluded that this approach would not require the determination of an undrained shear strength profile of the clay, although the determination of the depth to a hard stratum might still be required. He also suggested that slope stability plots could be developed to enable an estimate of stability which could be used for residential buildings whereby reducing the reporting costs for such structures. Although it is clear in his report that he believed the effective stress approach offered the best hope of understanding the behaviour of Winnipeg clays, he concluded that there were too many unknowns to support the use of this method.

Over the next 10 years, it would appear that the majority of stability analysis in the City of Winnipeg was carried out using total stress analyses. When effective stress analysis was used, peak shear strengths were often assumed for clays in which no previous failures had occurred based on the results of triaxial compression or direct shear tests. This method however, required that an assumption with respect to pore water pressures be made. Janzen (1972) studied the various methods to analyze Winnipeg riverbanks and concluded that the total stress analysis was not reliable as it was too empirical an approach. He also reported (citing Baracos 1961) that the total stress approach had not consistently overestimated the stability of riverbanks to the same degree as effective stress methods using peak strength values. With this in mind, he concluded that effective stress analysis for Winnipeg riverbanks should make use of residual rather than peak shear strengths where failures had occurred in the past. The importance of determining realistic pore water pressure distributions was also emphasized by Janzen and in follow-up work by Van Cauwenberghe (1972). Baracos (1978) reported that when measured pore water pressures, river levels and the geometry of the slip surface are taken into account, the application of residual shear strengths yields good correlations between theory and actual stability conditions.

Freeman and Sutherland (1974) compared the results of stability analysis using total stress methods with a shear strength of 500 psf and effective stress methods using residual shear strengths with circular and non-circular slip surfaces. Based on this comparison, they determined that the safety factor was substantially underestimated if residual strengths were assumed along the entire length of the slip surface and perhaps more importantly that the safety factor for slopes assuming a circular failure surface may be overestimated by as much as 0.5. They pointed to a tendency for the critical failure surface to be non-circular when the slope angle decreases, the depth to a hard stratum such as till is shallow, a weak stratum exists at the lower portion of the slope, and shear strengths are lower along horizontal layers.

The question of the strength of previously unfailed riverbank clays adjacent to failed areas (where residual strengths are applicable) was raised in the early 1980s (Baracos and Graham 1981). The authors concluded that the application of residual strengths everywhere along theoretical slip surfaces would underestimate the level of stability and the use of peak strengths would undoubtedly overestimate the level of stability. This conclusion was in agreement with previous research, notably the work by Rivard and Lu (1978) and Lefebvre (1981). This was the point in time where the concept of what was then called "fully softened strength" was introduced for unfailed portions of riverbanks; a conservative assumption of zero cohesion was made with an angle of shearing resistance between 16 and 23 degrees (Baracos and Graham 1981). Out of this work, came the recommended criteria for locating buildings on potentially unstable riverbanks as shown in Figure 7-9.

By the mid 1980s, it became common practice to carry out stability analysis using residual strengths in failed portions of the bank of c'_r = 3-5 kPa, ϕ'_r = 8-12 ° and post-peak[22] strengths of c' = 5 kPa, ϕ' = 17 ° for first-time slides. Piezometric levels were commonly taken at the ground surface with seepage parallel to the slope. It was found that if slopes were analyzed with zero cohesion (as might be expected for large strain or residual strengths), the critical slip surfaces tended to have a large radius and be close to the surface of the slope (Graham 1986). It was also found (and still commonly accepted), that the introduction of a small amount of cohesion forces the failure surfaces deeper and

[22] These strength values have also been referred to locally as ``fully softened``.

in better agreement with observations of slope movements. Assuming a small cohesion is therefore a way of approximating slightly curved failure envelopes in the moderately to highly plastic Lake Agassiz clay in Winnipeg.

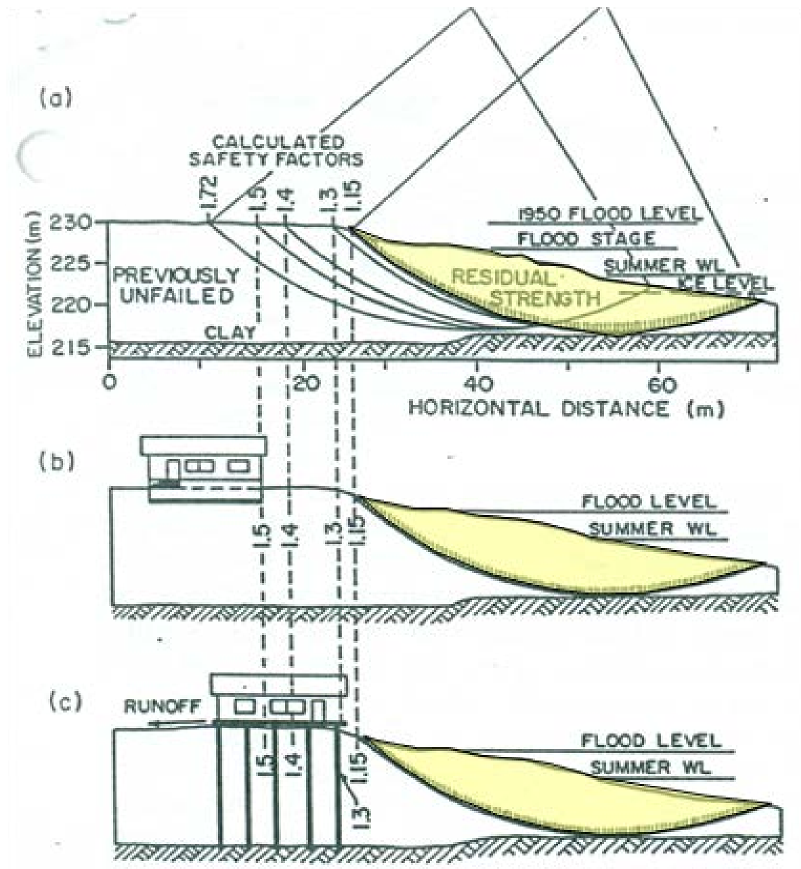

Figure 7-9 Results of effective stress analysis (Baracos and Graham 1981)

A study commissioned by the City of Winnipeg Rivers and Streams Authority in 1992 showed that groundwater pressures in the clay, till and bedrock aquifer can all significantly influence riverbank stability (KGS 1982). This work complemented the previous studies by Baracos (1978) and provided the means to input more complicated (but also more realistic) piezometric conditions in riverbank stability analysis. The major conclusions drawn from the study were:

- High upward flows occur in the lower clay in the fall, winter and spring (pre-flood) due to regional recharge from the upper carbonate aquifer. When this condition is combined with low river levels, a lower level of stability exists,
- Pore water pressures in the lower clay decrease due to consumptive use of the upper carbonate aquifer. When combined with regulated summer river levels, a higher level of stability exists,
- Heavy precipitation and bank saturation may critically lower the level of stability in the summer, and
- Riverbanks are most stable with a high river level, low piezometric level in the upper carbonate aquifer and no precipitation.

In 1990, a transient seepage model was incorporated into a slope stability analysis to assess the impact on riverbank stability associated with fluctuations in piezometric levels in the upper carbonate aquifer and river level (Tutkaluk 2000). Analysis was also carried out using a phreatic[23] groundwater elevation using shear strength parameters ranging from c' = 3 kPa, ϕ' = 8° (residual) to c' = 5 kPa, ϕ' = 17° (post-peak). For each set of shear strength parameters, it was found that methods incorporating groundwater gradients consistently yielded higher factors of safety than the model assuming hydrostatic groundwater conditions. The largest difference was observed when the bedrock aquifer is beginning to recharge and the river level is constant with corresponding increases in piezometric levels in proximity to the slip surface. The results were in close agreement when the bedrock aquifer was nearly completely recharged and the river level was constant. Decreases in the level of stability with assumed hydrostatic groundwater levels are almost entirely related to decreases in river levels.

[23] The natural static level of water in the ground. The magnitude of *pore water* pressure at the phreatic surface is zero.

Slope stability analysis has been typically done using a method called limit equilibrium analysis (Krahn 2003). Around the year 2000, a new approach was introduced in which finite elements were used for simulating both pore water pressures and stress distributions everywhere in slopes. These were then used to calculate ratios of available shearing resistance to shear stress demand along potential failure surfaces. The so-called 'generalized finite element method' was used for designing the expansion of the Red River Floodway (Skaftfeld, Kenyon and Van Helden 2009). An important benefit of the finite element method for analyzing stability compared with traditional limit equilibrium methods is the ability to identify progressive failure mechanisms. Traditional limit equilibrium methods assume averaged mobilized shearing resistance along the entire failure surface whereas the finite element stability methods calculates a distribution of mobilized shearing resistance, which is variable, along the failure surface. This solution is the most rigorous deterministic solution currently available for examining the global safety factor for assumed failure surfaces. The sensitivity of the proposed Floodway channel stability to potential variations in piezometric conditions was examined with respect to the following conditions:

- The observed historic maximum measured hydraulic potentials in the bedrock,
- The measured bedrock potentials during late fall/early winter of 2003/2004, and
- Piezometric conditions 2 m higher than even the observed historic maximum bedrock potentials.

Triaxial compression testing and consolidation testing were completed on soil samples collected from the locations of the channel and each bridge crossing as part of the first phase of investigations. Triaxial testing consisted primarily of consolidated undrained compression testing with pore pressure measurements, along with a lesser number of consolidated drained compression tests. Peak and large strain[24] effective deviator shear strengths for both the weathered brown clays and for the underlying unweathered grey clays were determined and compared with test data from the 1962 PFRA investigations. The results of the large strain testing are summarized on Figure 7-10. The selected design effective strength line forms a lower bound of all measured large strain deviator strengths. It also forms an approximate lower bound for virtually all the measured effective strengths for the brown and grey clays. As such, the selection of c' = 5 kPa, ϕ' = 14° forms an appropriate estimate of effective strengths for the purposes of Floodway stability modeling. These strength parameters have also become somewhat accepted in Winnipeg as values that can be used in riverbank stability analysis. Experience has shown that the measured peak strengths are significantly higher than the mobilized strengths in riverbanks and are not typically used for design.

It is becoming increasingly clear to geotechnical practitioners that the deterministic (safety factor) approach to slope stability analysis fails to systematically assess and account for the various sources of uncertainty in establishing representative soil and groundwater conditions (Van Helden 2013). The safety factor does not contain any information regarding the level of conservatism, uncertainty or reliability. To illustrate this point, research was carried out to provide an estimate of the probability of failure for the channel side slopes for the Floodway expansion to relate the reliability of the channel to the reliability of the flood protection system as a whole. The study involved the

[24] Following reaching "peak strength", inter-particle bonding continues to weaken and the clay strain softens. Once all bonding has been destroyed, shearing resistance reaches a "steady state" condition of constant deviator stress and stress ratio known variously as "post peak", "undrained strength at large strains" or "normally consolidated" shear strength.

development of a deterministic slope stability model as well as statistical distributions for its input parameters, in particular, large strain stress states from the triaxial testing. The estimated annual probabilities of channel slope failure were determined for three unique cross sections. The results showed good agreement with the deterministic approach in identifying the critical cross section. The application to riverbank stability will be acceptable providing sufficient data is available to economically and in a reasonable time frame, to support its use.

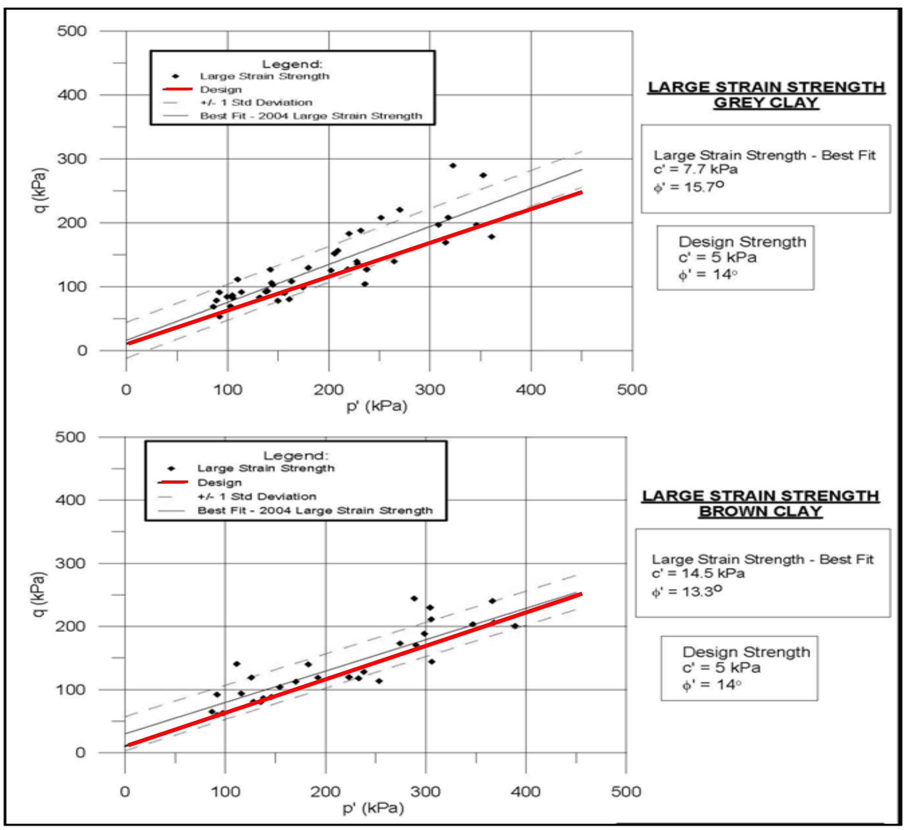

Figure 7-10 Summary of large strain effective strengths (KGS, ACRES, UMA 2004)

7.4.2 Current State-of-Practice

The preceding section describes how over the last few decades, the state-of-practice in riverbank engineering has evolved in Winnipeg from the highly empirical use of total stress analysis and circular failure surfaces to one that uses effective stress analysis that fully couples stress-deformation and groundwater flow in transient or steady-state conditions. It is important however, not to lose sight of the value of the ***observational*** approach regardless of the analytical tools used to solve a problem. This was the approach used for the original design of the Floodway, a decision which ultimately has been proven to be both technically correct and economical.

The preceding section also highlights the significant information that has been gathered with respect to soil strength and groundwater conditions. While there is no doubt that the properties of lacustrine clays have been very well researched, in particular during the design phase of the expansion of the Red River Floodway, there is little published information relative to alluvial soils. This is perhaps surprising given that significant stretches of riverbanks are comprised of alluvial soil or alluvial soils overlying lacustrine soils. An understanding of the strength properties of both the lacustrine and alluvial soils is therefore important for properly characterizing riverbanks for stability analysis.

The information gathered on groundwater conditions has provided a means to assess riverbank stability during different times of the year, an ability which has lead to the determination of the level of stability for "worst case" or "short term extreme" conditions and "typical" or "long term" conditions. Short term extreme conditions are typically associated with late fall/early winter when river levels are low (under drawdown conditions) and piezometric levels in the till (often hydraulically connected to the upper carbonate aquifer) remain high. The target safety factor under these conditions is typically 1.3. Typical conditions generally reflect regulated summer river levels and average piezometric levels in the till and the target safety factor is typically 1.5. Aside from the potential for bank erosion, the recession of high water levels in late spring (to the regulated summer river level) is not usually the most critical time with respect to slope stability. Depending on the design, variations to these conditions can be applied. However, the approach outlined in earlier sentences is widely accepted and is often the required approach for infrastructure improvement works such as flood protection.

The results of stability analysis are very much dependent on the selection of soil properties, in particular the shear strength as defined by cohesion (c') and the angle of internal friction (ϕ'). Recently, there has been a departure from the use of shear strength values of c' = 5 kPa, $\phi' = 17°$ to the large strain shear strengths of c' = 5 kPa, $\phi' = 14°$ for unfailed portions of the bank based on the Floodway testing. Failed portions of the bank are often assigned residual strengths which typically are back-calculated based on an assumption that the existing safety factor for the failed portion of the bank is unity (1.0). The back analysis typically assumes near zero cohesion with $\phi' = 8\text{-}10°$ although larger friction angles are often calculated for alluvial soils.

It is the author's experience that both sets of shear strengths are applicable. Many successful projects have been undertaken with stability analysis based on shear strength values of c' = 5 kPa, $\phi' = 17°$, typically though with an assumed phreatic groundwater level. If groundwater gradients are incorporated with shear strengths of c' = 5 kPa, ϕ' =

14°, the calculated factors of safety and slip surface geometry in lacustrine clay riverbanks are in close agreement.

It is generally accepted that for normally consolidated clays, the real cohesion (c') is negligible and the best fit line for test results is $c' = 0$, $\phi' =$ constant (Trainor 1982). Strength envelopes for Winnipeg (lacustrine) clays however are typically curved or bilinear, with ϕ' varying from about 24° at $p' = 200$ kPa to 18° at $p' = 500$ kPa. This behaviour is unusual and perhaps related to the mineralogy of Winnipeg clays. The question arises as to whether the assumption of a cohesion intercept of 5 kPa is valid.

As mentioned previously, it appears to be necessary to assume a small amount of cohesion in order for theoretical slip surfaces to match more closely the observed behaviour of riverbanks. This comparison however, is generally made with the case where $c' = 0$ and $\phi' =$ a constant value. It is of interest therefore to examine the results of stability analysis where ϕ' varies with confining pressure, that is, a curved failure envelope. Although it is now possible to fully define the shape of the failure envelope in some slope stability models, it may be reasonable to break the failure envelope (or CSL line in p' q space) into two segments, essentially defining a bilinear strength envelope. An interesting possibility is that at low confining pressures, for example near the crest or toe, the shear strength of the soil may be overstated if a linear extension of the stress-strain curve is drawn to intercept the vertical axis at zero confining pressure. This is illustrated in Figure 7-11.

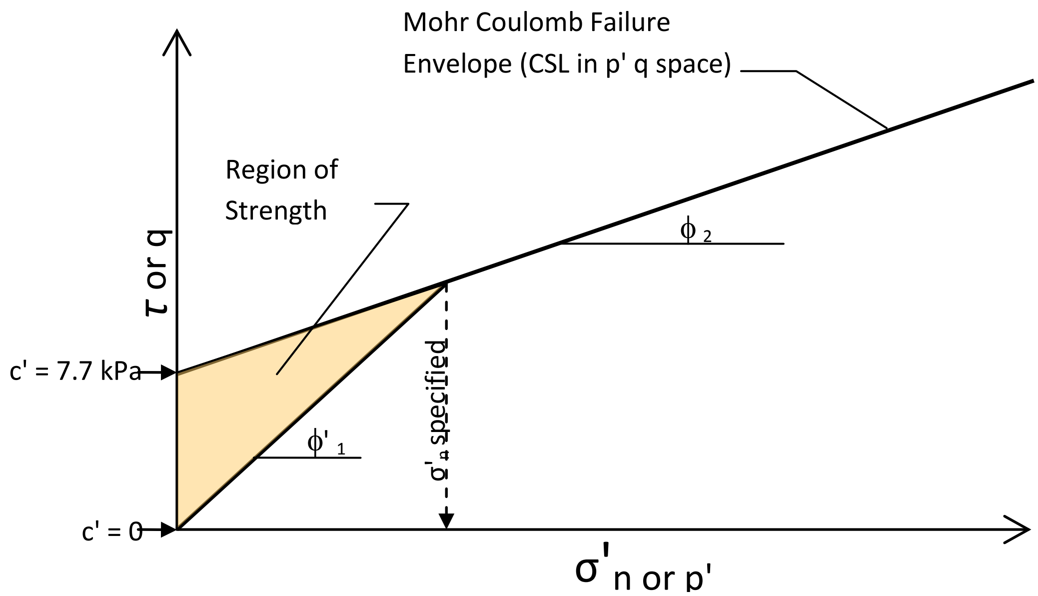

Figure 7-11 Bilinear strength envelope

If c' and ϕ'_2 are known and it is assumed that there are two unique but overlapping failure envelopes (linear and bilinear), ϕ'_1 can be calculated by assuming the strength equations (τ = c' + σ'_n tan ϕ') for the two envelopes are equal at the specified normal stress (σ'_n). The shaded area between the envelopes is referred to as the region of strength differential and represents the range of normal stresses where the calculated shear strengths will differ. This may also be the region where σ_3=0, that is, where the soil is in tension (Baracos, Graham, Domaschuk 1980). Take for example, the best fit line from Figure 7-9 for grey clay (c' = 7.7 kPa, ϕ' = 15.7°); if σ'_n is assumed to be 50 kPa, then ϕ'_1 = 19.3°.

The author has compared a linear and bilinear strength envelope to determine the impact on the calculated safety factor and the geometry of the critical failure surface for a simple riverbank model. The bank height is 11 m high with a slope of 4H:1V. One layer of lacustrine clay over till is assumed and the till contact is deep enough to prevent the development of a composite slip surface in the model (i.e. the theoretical slip surfaces will be circular). The (minimum) safety factor associated with the critical slip surface is 1.065 for the model with the linear strength envelope (c' = 5 kPa, ϕ'_{nc} = 14°) as shown in Figure 7-12. If a bilinear strength envelope is assumed (ϕ'_1 = 19.3°, ϕ'_2 = 14°), the calculated safety factor for the critical slip surface is 1.009, however, this slip surface is somewhat shallower.

In order to force this slip surface deeper to more closely match the geometry of the linear soil strength model, soil suction (negative pore water pressure) was incorporated. With a ϕ^b value[25] of 10°, the safety factor associated with the critical slip surface is 1.040 and the slip surface geometry is nearly identical to that determined using the linear model (Figure 7- 13). While the same relationship may not exist for other combinations of stengths or slope geometries, it does suggest that the application of unsaturated soil in riverbank stability analysis should be investigated further, in particular if the use of a bilinear strength envelope is under consideration.

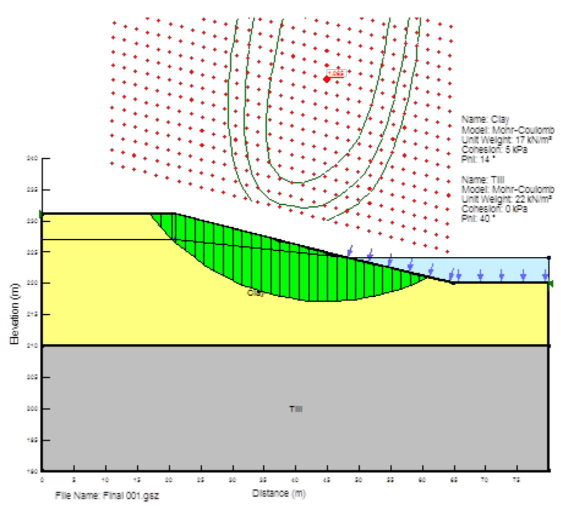

Figure 7-12 Linear strength model

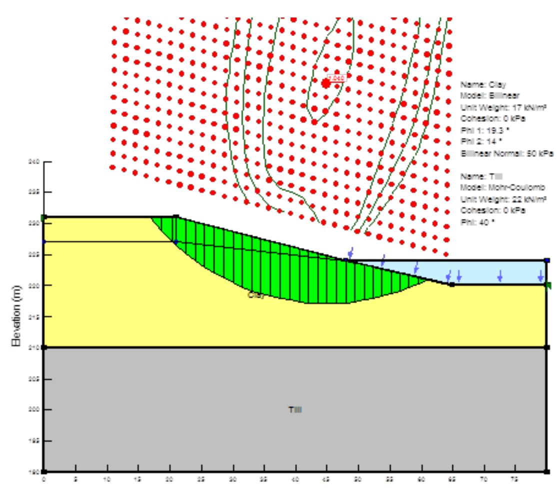

Figure 7-13 Bilinear strength model

[25] The term a ϕ^b is a soil property and often assumed to be about 1/2 ϕ'

The concept of unsaturated soil mechanics can be rationalized by considering what a realistic seepage boundary condition would be for Winnipeg clays: work by Vaughn (1994) has shown that in soils with a hydraulic conductivity of less ahan 10^{-6} cm/sec, capillary rise would be significant and the assumption of a phreatic surface will be invalid. Depending on the nature of evaporation and recharge from precipitation, there can be a considerable thickness of desiccated soil or a relatively high water table with corresponding cyclic changes in pore water pressure and effective stresses near the ground surface as a result of wetting and drying. For example, Vaughn (1984) determined that the average surface suction for UK clays is about 10 kPa which is equivalent to a groundwater table 1 metre below ground surface. In Winnipeg, the most likely time where hydrostatic conditions would exist would be in the spring after cracks created in say a dry fall are filled with water (the upper desiccated layer becomes nearly saturated). In a dry summer or fall, the effects of soil suction are likely real and perhaps the introoduction of negative pore water pressures near ground surface is valid.

In each of the models examined by the Author, the portions of the slip surface associated with c' = 0 kPa, ϕ'_1= 19.3° are at the crest and toe where effective stresses are less than 50 kPa (about 3 slices). The strength parameters along the remainder of the slip surface are based on c' = 7.7 kPa, ϕ'_2 = 15.7°. If for example however, a confining pressure (σ_n) of 80 kPa is assumed, the entire length of the slip surface is defined by the strength parameters c' = 0 kPa, ϕ'_1= 19.3°. Careful consideration should therefore be given to the determination of the inflection point for the bilinear strength envelope relative to the depth of the slip surface.

Often one of the most challenging problems is to differentiate alluvial soils from lacustrine soils and assign the appropriate shear strength properties. Even on an inside bend where alluvial soils would be expected, it may be difficult to determine the contact with lacustrine clays and straight or transition channel sections (from an inside to an outside bend) could be either alluvial or lacustrine. Detailed visual classification and laboratory testing from samples collected along Winnipeg riverbanks have been compiled by the Author to assist in identifying depositional history based on soil index properties.

The percentage of clay size particles (< 2 microns) and the liquid limit have been plotted against the plasticity index on Figures 7-14 and 7-15 respectively. Based on a visual classification of the soil samples supplemented with riverbank characterization, it would appear that the boundary between alluvial and lacustrine clays in Winnipeg is at a plasticity index of about 40%. This relationship should be used with caution, in particular where plasticity indices lie close to the division. The results from Atterberg testing are summarized on the plasticity chart on Figure 7-16 which also shows the approximated boundary between alluvial and lacustrine soils.

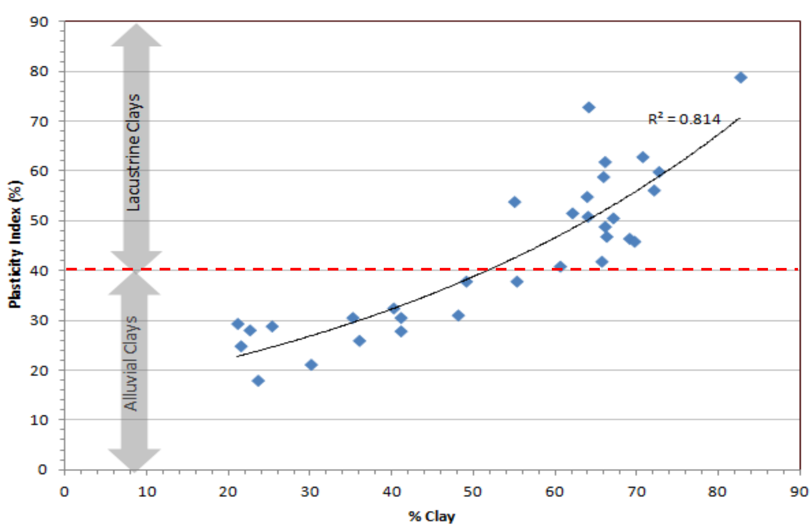

Figure 7-14 % Clay vs. Plasticity Index

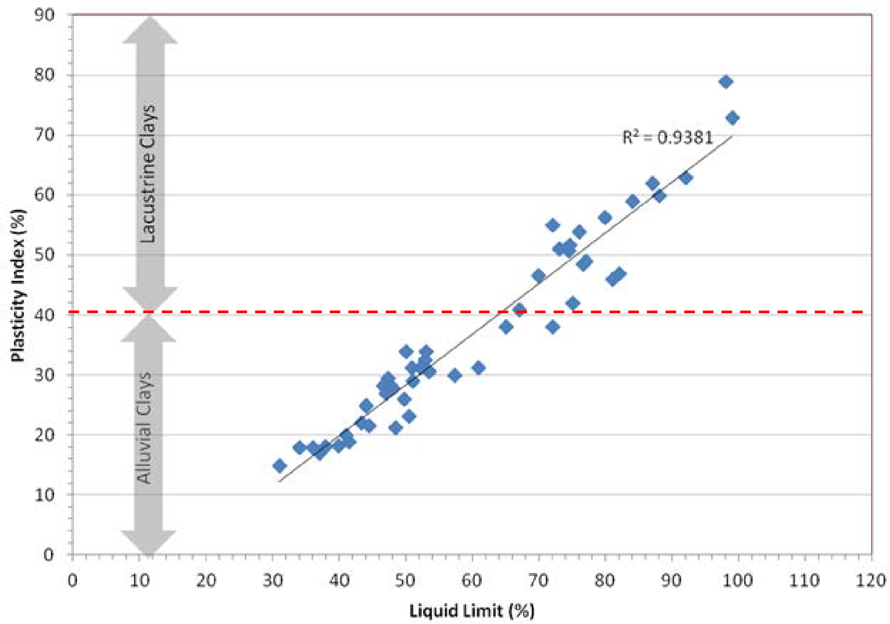

Figure 7-15 Liquid Limit vs. Plasticity Index

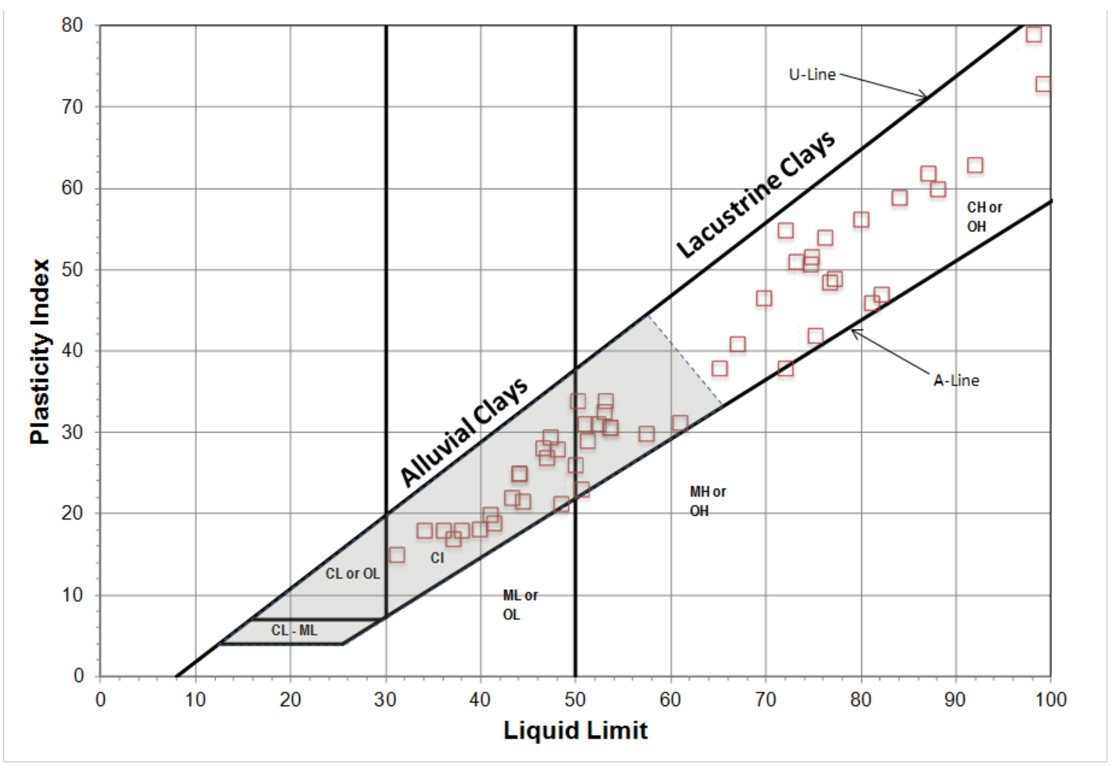

Figure 7-16 Plasticity chart for riverbank soils

Data available in the literature clearly show a trend of reducing ϕ'_{nc} with increasing plasticity index (I_p) (for example, Sorenson 2013). The plasticity indices measured on soil samples from various sources have been plotted against large strain (normally consolidated) friction angles (ϕ'_{nc}) from triaxial compression and direct shear testing on Figure 7-17. The data set includes Lake Agassiz clays (primarily along Winnipeg riverbanks) and results from other locations outside of Winnipeg. Also shown are the mean and lower bound curves from various published data sets and strength results from relatively pure specimens of kaolinite, illite and montmorillonite. The boundary between alluvial and lacustrine soils as previously interpreted, has been also been approximately overlaid onto the plot of strength results.

Figure 7-17 shows a reasonably well defined trend towards decreasing friction angle with increasing plasticity, with all of the data from Winnipeg falling between the mean and lower bound curves derived from published data sets. It would also appear that clay mineralogy plays a role in strength, with illite (which constitutes about 45% of the clay)[26] overlapping the boundary between alluvial and lacustrine clays. Based on these results, a value ϕ'_{nc} of 14 degrees appears to be a reasonable lower bound value for all lacustrine clays while $\phi'_{nc} = 16°$ is reasonable as a mean value. Alluvial soils can be bracketed into two ranges of plasticity; an I_p 10% to 30% and from 30 to 45%. The corresponding effective friction angles would be from 25 to 30° and from 20 to 25°.

[26] Winnipeg clay is estimated to consist of 45% illite, 35% montmorillonite (layered with illite), 5-10% kaolinite, and 10-15% quartz (Quigley 1968).

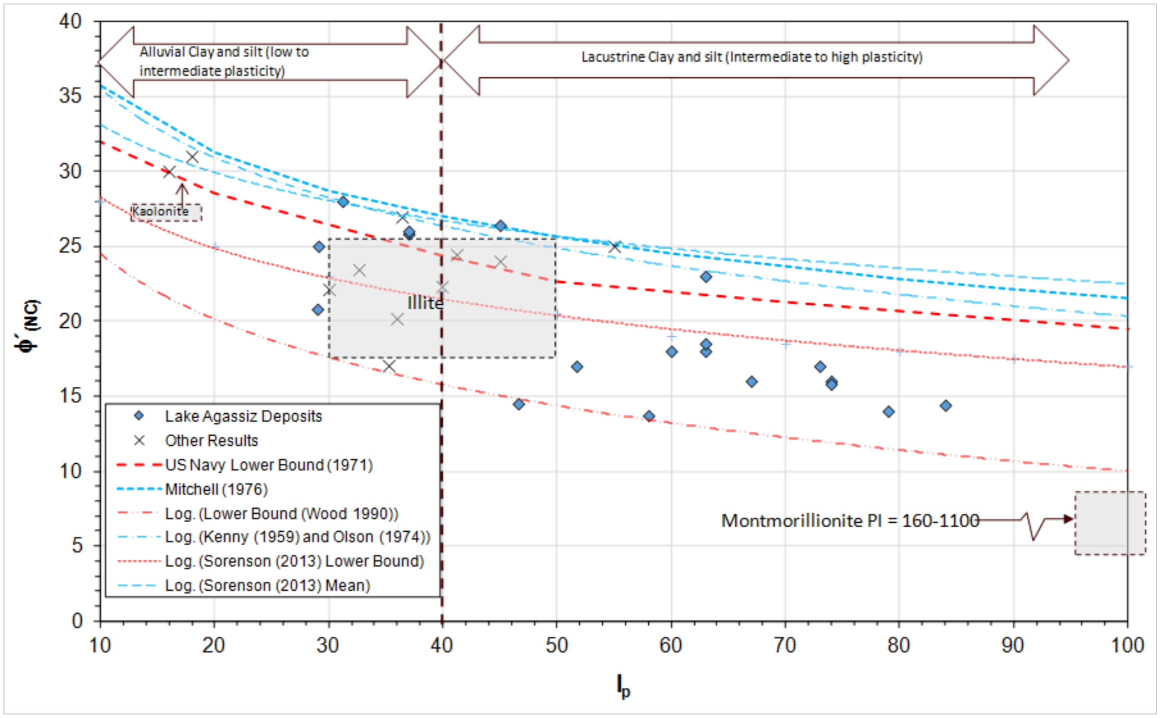

Figure 7-17 Relationship between Plasticity Index and normally consolidated friction angle

7.5 METHODS OF STABILIZATION

In the last 50 or 60 years, many methods of riverbank stabilization have been tried and in many cases, improved upon in the City of Winnipeg. Early methods were often based on experience and judgement with (apparently) little or no stability analysis. These stabilization measures often involved trimming the bank to a stable slope and then protecting the lower bank from erosion. Erosion protection measures included gabion baskets, timber pile walls along the mud line of the river and driven to the till (along the Assiniboine River in particular), and riprap. It was also recognized however, that these measures would have little value in preventing deep-seated slides in the overall riverbank.

Robert M. (Bob) Hardy became involved in stabilization projects in Winnipeg in the 1960s. On several of his projects, timber ribs running perpendicular to the river were designed, and at least one project was constructed at the Shriner's Hospital for Children (633 Wellington Ave.) in 1962. The working theory for the ribs (spaced at approximately 9 m) was they acted as stiffening members by transferring shearing stresses from the soil to the ribs by skin friction between the soil and surface of the ribs. The long term performance of these measures is not known.

In the early 1980s, considerable progress was made on riverbank stabilization using sand drains installed perpendicular to the bank (Lew and Graham, 1988). The drains were typically 0.6 to 1 m wide, spaced 5 to 10 m apart and extended to about 5 to 6 m below existing grade. Augered drains could be used to effectively extend these to greater depths. The primary design intent was to lower groundwater levels thereby increasing effective stresses and hence stability. Improvements of 15% were reported for cases where the groundwater level in the clay could be lowered by 2 m with a further improvement of about 15% through the mechanical strength of the sand.

A significant factor affecting the performance of the drains is the hydraulic conductivity (permeability) of the clay; the design is based on a mass permeability of 10^{-7} and 10^{-8} m/s. Although the permeability measured on intact samples typically ranges from 10^{-10} to 10^{-11} m/s, it was recognized that the upper 4 m of clay is heavily fissured and therefore has a mass permeability several orders of magnitude higher (Graham and Shields 1984). Whether the permeability of the clay was sufficiently high to provide the necessary reduction in pore water pressures is debatable as there was no direct measurement of pore water pressures between the drains in two reported case studies (Lew and Graham 1988). Pneumatic piezometers installed in 1993 between sand drains installed at the Mager Drive Pumping Station however, did not indicate a significant lowering of piezometric levels over the duration of monitoring (Author's personal account). Other factors negatively impacting on performance are thought to include the (limited) depth of installation and limited length relative to the position of the head scarp and toe (pers. comm. J. Graham).

The contribution of mechanical improvement recognized for sand drains was carried forward in the concept for granular ribs which also provide internal drainage (although usually not accounted for). By the early 1990s, local practitioners began favouring granular shear keys, granular ribs and more recently, rockfill columns. Shear keys are constructed by excavating a trench along (parallel to) the toe of the slope, through the failure surface and into the till. The trench is then backfilled with granular fill (generally crushed limestone). Granular ribs consist of a series of parallel trenches excavated into the riverbank but perpendicular to the toe and into the till and backfilled with crushed limestone. Rockfill columns are large diameter holes drilled in rows parallel to the toe of the slope (either mid-slope or at the toe) and into the till which are then backfilled with crushed limestone.

All of these methods have the same operating principle whereby the weak (failed) clay is replaced with a higher strength granular fill essentially creating a shear wall (Yarechewski and Tallin 2003). The physical dimensions of the works depend on the assumed friction angle and desired level of improvement. Granular ribs and rockfill columns have the advantage of allowing only a limited portion of the lower bank to be opened up at one time before backfilling, thereby greatly reducing the risk of bank movement during construction compared with a shear key.

Granular shear keys are generally limited to sites where the till is within 8 m of ground surface (Figure 7-18). The base width of the shear key is generally 3 to 4 m with walls kept as nearly vertical as possible. Granular ribs are often up to 15 m in length, up to 8 m deep and spaced 1 to 3 m (face to face) with near-vertical excavation walls (Figure 7-19). Caving can be problematic for close spacing and so it is often desirable to increase the length of the ribs and maintain spacing closer to 3 m center to center. Good compaction is absolutely essential to the success of these forms of construction. For both shear keys and granular ribs, it is generally accomplished using a vibratory plate mounted to the end of an excavator arm or a vibratory lance attached to an excavator (Figure 7-20).

Figure 7-18 Shear key construction

Figure 7-19 Construction of granular ribs

Figure 7-20 Compacting rockfill in shear key. *Compaction is achieved using a vibratory probe mounted to an excavator on a grid pattern.*

Hybrids of these methods include the addition of tie-back anchors to the shear key along Lyndale Drive to increase the safety factor from about 1.3 with only the shear to 1.5 with the anchors (Figure 7-21). At the Mager Drive Pumping Station, granular ribs were installed to temporarily increase the safety factor to about 1.15 prior to excavating the shear key on the downslope side of the ribs to achieve a final SF of 1.4 was (Figure 7-22).

Figure 7-21 Installation of tie-back anchors in shear key

Figure 7-22 Vibratory plate used to compact rockfill for shear key. *The Granular Ribs (installed before the Shear Key) Can Be Seen at the Edge of the Shear Key*

Compared to granular shear keys and ribs, rockfill columns can be installed to greater depths, although casing may be required if sloughing of the clay or wet till conditions are encountered. This can greatly add to the construction cost. Recently, rockfill columns were successfully installed to depths of 25 m for stabilization of the east and west riverbanks at the PTH 23 bridge over the Red River in Morris Manitoba. This job required installation of full-length sleeves which were vibrated out of the shafts after crushed rock was placed, thereby compacting the lower portion of the column (Figure 7-23). Typically, two or more rows of rockfill columns are installed with a common (most economical) diameter being 2.1 m with face to face spacing as close as 0.6 m. Rows are typically staggered to provide an overlap of granular material. Drilling is performed with an auger rig and compaction is typically performed with a vibratory lance (Figure 7-24).

Figure 7-23 Installation of 25 m long rockfill columns on Red River. *A vibratory unit (right hand photograph) was used to remove the casing and at the same time, compact a portion of the rockfill.*

Figure 7-24 Drill rig auger (left photo) and compacting rockfill columns with Vibrolance

Granular shear keys are designed by incorporating a simple keyway in a two dimensional slope stability model. The width of the key is varied until the desired level of improvement is achieved. Granular ribs are typically modelled as a shear key with shear strength properties of a composite soil based on an assumed trench width and spacing. In this fashion, the length (width in the model) of the ribs, the rib width and spacing can be optimized. Rockfill columns are designed based on an equivalent shear key width with the area of the shear key (in plan view) converted to an equivalent area of circular columns. For example, a 3 m wide shear key is equivalent to 2 staggered rows of 2.1 m diameter rockfill columns spaced at 0.6 m face to face.

Early shear key designs assumed friction angles of 35° for the friction angle of the rockfill. These design values were later increased to 45° based on the results of large scale direct shear testing carried out by UMA Engineering (1986) which showed the friction angle may be as high as 55°. This measured increase in friction angle allowed the size of stabilization works (for example the width of a shear key) to be reduced by nearly 50%. The direct shear test apparatus and test results compared with published results are shown on Figure 7-25.

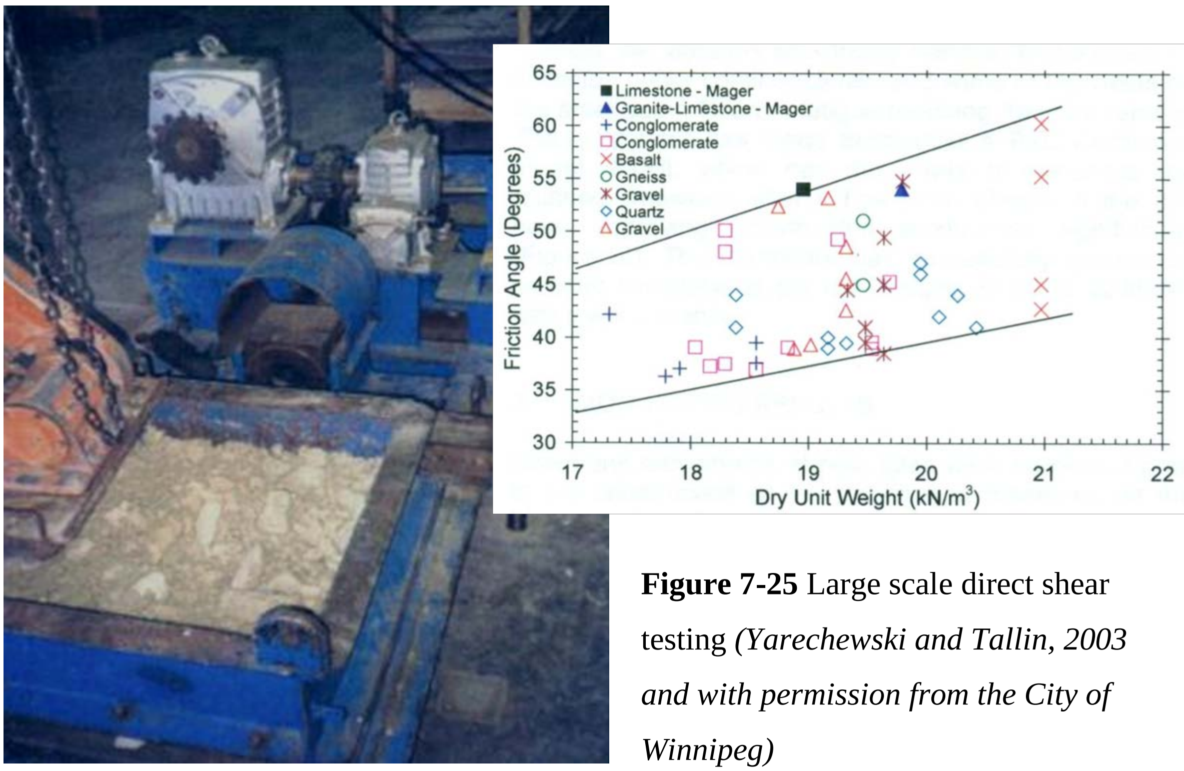

Figure 7-25 Large scale direct shear testing *(Yarechewski and Tallin, 2003 and with permission from the City of Winnipeg)*

The importance of compacting granular fill is well known. Based on recent large scale direct shear testing at the University of Manitoba (Abdul Razaq 2007) and a full scale test site (Thiessen 2007) friction angles of 50° or higher are now being used for design although it is imperative that good quality control be exercised during construction if these values can be relied upon. In the Author's opinion, friction angles exceeding 50° may be achievable in the laboratory but cannot be relied upon with confidence in the field.

Compaction techniques for rockfill columns have improved considerably over the years based on the understanding that the friction angle is dependent on the bulk unit weight and that post-construction movements can be greatly reduced with well compacted backfill material. In the early years, granular fill was dumped loose into the excavations or shaft with no compaction other than what could be archived after free-falling. Field tests determined that the maximum dry unit weight of frozen crushed limestone (most stabilization works are carried out during the winter) was about 16 kN/m^3 for loosely dumped material which is considerably less than required to achieve a friction angle of 45° (refer to inset on Figure 7-25). Field trials during the construction of stabilization works for the Mager Drive Pumping Station showed that frozen crushed limestone could successfully be compacted to dry unit weights of about 18 to 19 kN/m^3 using a vibratory plate packer mounted on an excavator arm, a technique which was used successfully on several projects.

Compaction of rockfill columns was first attempted using a drop hammer with limited success. During construction of stabilization works for the Branch I Aqueduct crossing at the Seine River, a compaction tool was devised by Subterranean (Manitoba) Ltd. which consisted of a 15 m long steel beam with four steel fins weeded to the end (Figure 7-26). The device was lowered to the bottom of the shaft before it was backfilled with loosely dumped granular fill. The column was then vibrated and extracted using a crane mounted vibratory pile driving hammer mounted to its top. Dry densities ranging from 17.5 - 18.8 kN/m^3 were archived using this method. Although this was successful, the device required frequent repairs and it was subsequently replaced with Vibrolance® which was capable of achieving in place dry unit weights up to 22 kN/m^3. The lance penetrates the rockfill columns after backfilling and is then slowly withdrawn. Water may be added to the granular backfill to assist in compaction and help cool the lance. Specifications typically call for a 15% increase in density from the loosely placed to compacted condition. Compliance is determined by measuring the amount of drop in the rock after compacting and calculating this as a percentage of the initial height (for example, rock fill in a 12 m long columns would drop by 1.8 m) as shown on Figure 7-27.

Figure 7-26 Vibrating steel columns to compact rockfill

Figure 7-27 Rockfill dropping during compaction

As rockfill column compaction methods were being developed, there was some concern that the apparent amount of compaction, as determined by the drop in the rock, was partly due to outward bulging of the shaft, particularly in the soft grey clay at depth, thereby resulting in an incorrect (overstated) in-place unit weight. UMA Engineering evaluated this concern by drilling a test column in the center of an array of three slope inclinometers at a test site in Subterranean Ltd.'s yard in northern Winnipeg. Horizontal displacements were measured after drilling and immediately after backfilling and compacting the rock fill with the lance. The unpublished results showed very small inward movement (contraction) of the shaft after drilling and essentially all of this deflection was recovered after backfilling and compaction with little to no outward bulging. It was concluded that the drop in the rock was therefore attributable entirely to densification of the granular fill.

It is also worth noting that construction experience has shown that excessive vibratory compaction may result in mechanical break-down of the aggregate and should therefore be avoided.

7.5.1 Regulatory Considerations

Waterways

The Waterways Section of the Planning Property and Development Department of the City of Winnipeg regulates construction along about 150 kilometres of waterways within the city under an application and permit process. This regulation goes back to the aftermath of the 1950 flood when many uncontrolled activities lead to riverbank failures and waterway obstructions. This situation led to the creation of the ***Rivers and Streams Authority No. 1*** in 1951 under Part III of the Rivers and Streams Act. Section 422(3) of the City of Winnipeg Act stated that City Council was the Authority for "The Winnipeg Rivers and Streams Designated Area No. 1". City Council then delegated its authority to consider applications and approve permits to the ***Rivers and Streams Committee***. All approvals of the Rivers and Stream Committee were subject to the confirmation of the Committee on Environment prior to the issuance of a permit under the Rivers and Streams Act. In general, the purpose of the Authority was to ensure:

- That no deposit shall be placed on the riverbanks which will have the effect of restricting or impeding the flow of water; and
- That no deposit shall be placed or building constructed which will endanger the stability of the riverbank or cause any part of the riverbanks to slip into the channel.

The jurisdiction of the Authority initially extended 150 feet (45 m) from the Red and Assiniboine Rivers only. The legislation was subsequently amended to include properties within 350 feet (107 m) of the normal summer water mark of the Red, Assiniboine, Seine and La Salle Rivers; and 250 feet (76 m) of Omand's, Bunn's Sturgeon and Truro Creeks. Since 1992, waterway construction has been regulated by the City's Waterway ***By-law No. 588/92*** which was amended in 2002. The regulated area under Waterway By-law 588/92 is shown on Figure 7-28.

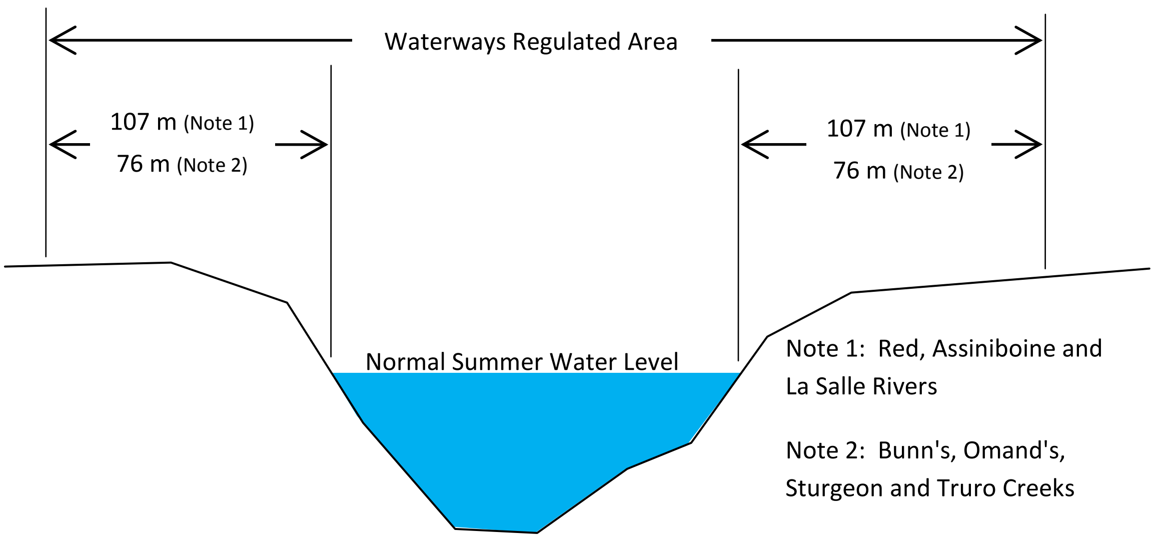

Figure 7-28 Regulated area under Waterway By-law 588/92

Waterway By-law 588/92 states that a permit shall not be issued for work to be done in a regulated area unless the person applying demonstrates to the reasonable satisfaction of the Director[27] that the proposed work will not, or will not have a tendency to:

- Restrict or impede surface or subsurface flow;
- Endanger the stability of any land, including the bed of a waterway;
- Cause land to slip into a waterway; or
- Adversely alter the channel of a waterway.

[27] "Director" means the Director of Planning Property and Development and his or her delegate; generally the Waterways Engineer

Before 2011, the Waterways Engineer reported directly to the Riverbank Management Committee made up several City councillors with one councillor acting as the Chairperson: For many years this was Councillor Harry Lazarenko. On March 23, 2011, Council passed By-law No. 48/2011 amending the City Organization By-law No. 7100/97 and other by-laws to implement jurisdictional changes for the Standing Policy Committee on Downtown Development, Heritage and Riverbank Management. In doing so, the stand-alone Riverbank Management Committee was disbanded.

In preparing an application for a Waterways permit, it is helpful to know what is considered necessary information and what conditions are likely to be imposed in the permit. While dependent on the size and complexity of the project, there are basic considerations that should be included in the submission if geotechnical support is required, including (but certainly not limited to) the following:

- Quantify the impact (positive or negative) associated with the work and clearly identify the target level of stability. If appropriate, determine the safety factor under worst case (extreme) conditions and typical conditions. Describe the measured or assumed parameters used for effective stress modelling, in particular piezometric levels and shear strength parameters.
- Define property lines and ensure work areas do not extend on to private (or public) properties unless written authorization can be provided. This includes any necessary site access or egress. On many river lots, the determination of the property line closest to the river is often defined as the ordinary high water mark (OHWM). The channel beyond the OHWM is Provincial Crown Land. The reality of this definition is that this portion of the property line may shift due to erosion or deposition of material. The final determination of the OHWM must be made by a Manitoba Land Surveyor.

- Identify material stockpile locations which will not adversely impact bank stability.
- Identify measures to provide smooth transitions for riprap and provide an assessment of impacts on channel hydraulics. Often this requires input from a qualified hydraulics engineer.
- Identify any precautionary measures to be employed during construction. For example, a shear key should not remain open overnight or when construction activities are interrupted.
- Identify necessary erosion and sediment control measures (refer to Best Management Practices Handbook for Activities In and Around the City's Waterways and Watercourses (available through the City of Winnipeg).
- If leaks from in-ground pools could adversely impact on riverbank stability, identify measures to minimize leakage and capture/redirect any seepage water, for example, perimeter drains and catch basin.
- Where applicable, provide confirmation that other regulatory agencies such as the Department of Fisheries and Oceans (DFO have been advised of the work.

Environmental Legislation and Guidelines

Work along waterways that supports or has the potential to support fisheries and aquatic resources is subject to all applicable environmental legislation, including federal, provincial and municipal legislation, guidelines and by-laws. The environmental legislation and guidelines applicable to works in and around a waterway and watercourse are summarized in the Best Management Practices Handbook For Activities In and Around the City's Waterways and Watercourses.

8 LESSONS LEARNED, CONCLUSIONS, AND RECOMMENDATIONS

8.1 LESSONS LEARNED

An understanding of the relationship between geology, groundwater, and geotechnical properties is essential for projects that involve site characterization and geotechnical design. While each of the three components can be broadly defined over a large area, it is generally necessary to assess engineering properties in a smaller context through detailed site investigations. Still, site investigations provide a representatively small sampling of the actual sub-surface conditions which must be interpreted three-dimensionally. This interpretation can be aided by local experience and information from other site investigations that may have similar geology and groundwater conditions. In this regard, there have been thousands of site investigations made locally by hundreds of practitioners. The experience gained by others can provide valuable supplemental information although should never be considered as a replacement for a site specific investigation.

The importance of erosional and depositional processes that change landforms has become increasingly obvious to the author over the years. An understanding of the geologic characteristics, the origin of earth materials and geomorphological processes can greatly assist in the determination of appropriate geotechnical properties, in particular along waterways within the city. In this regard, an appreciation of river morphology will assists in characterizing a riverbank to determine the most likely failure mechanism and allow for more strategic planning for subsurface investigations. For example, the presence of alluvial soils would likely require drilling techniques suitable for wet silts and possibly sands. It would also be essential to determine the contact (if any) with lacustrine soils which is of likely significance in the geometry of slip surfaces.

Groundwater can have a significant adverse affect during construction and on completed engineered works. Levels can vary seasonally or as a result of heavy precipitation (through infiltration), leaking utilities or construction activities. Of particular concern is evidence that there is a gradual but steady rise in groundwater levels in central Winnipeg associated with a sizable reduction in groundwater consumption. Rising groundwater levels are likely to cause problems such as base heave of basement floor slabs or excessive seepage during construction and they may also impact projects completed many years earlier. An associated concern is the potential for increased pore water pressures at the clay-till interface, reduced shear strengths, and therefore deep seated instabilities.

Geotechnical engineers are responsible for obtaining and interpreting subsurface information to be used in the design and construction of various civil engineering structures. The interpretation of site investigations and laboratory testing can directly impact the design in terms of conservatism and construction in terms of delays and potential for claims. The costs of an inadequate site investigation can therefore be considerable. Good site investigations are costly but the costs can readily be recovered in terms of more confidence in design decisions. A lot of money can be spent on a poorly planned investigation, for example a regular grid of holes compared with investigations targeted according to observed or anticipated sub-surface conditions. A simple site investigation with good interpretation is better than a detailed investigation with poor interpretation.

A good understanding of site investigation techniques, design principles, and construction methods is necessary. Once the project requirements are known, a geotechnical engineer should determine the site specific requirements for a sub-surface investigation. It is important to understand that changes to the sub-surface investigation may be required as information becomes available and that these changes may result in additional field costs. However, the costs of additional site investigations are usually small in comparison to the costs associated with uneconomical designs or construction delays. Any unusual or unexpected conditions should be relayed to the design team as quickly as possible as well as the justification for any changes in the sub-surface investigation.

Once the sub-surface investigation has been completed, laboratory testing should proceed as soon as possible to avoid sample deterioration. This is the time when the geotechnical design engineer should take advantage of the opportunity to see the samples and gain a hands-on appreciation of the soil properties. The importance of this step cannot be overstated. Once a feel for the natural variability of the soils has been developed, the results of the laboratory testing, modelling and design values will become clearer.

Shallow foundations such as footings for lightly loaded structures are often within the "upper complex zone" in Winnipeg, where there is a likelihood of variable soil deposits, in particular silt, which are subject to environmental effects from wetting, drying and freezing. Shallow foundations within the upper soils horizon are therefore likely to experience settlement or heave and the magnitude of this may be damaging. For this reason, many settlement-sensitive structures are supported on deep foundation systems.

Prior to the mid to late 1900s, most structures in downtown Winnipeg were in the three to eight storey range supported on heavily loaded footings or caissons advanced to till or bedrock. In the late 1960s, a skyscraper boom was experienced with the construction of the Richardson Building, Holiday Towers and Grain Exchange Building. Between 1980 and 1990, construction of high-rise buildings became common place, with the construction of the TD Centre (now Canwest Place) and Evergreen Place Towers on the Assiniboine River. Following a brief lull, numerous high-rise construction projects were initiated including Manitoba Hydro's downtown office tower and the Canadian Museum for Human Rights. While the height of these high-rise structures in Winnipeg has not changed significantly over the years, the foundation loads have dramatically increased as architects design buildings with more open space. This has greatly increased the reliance on good characterization of site geology, in particular the nature of the till and underlying bedrock units. It has also increased pressure from developers to provide more economical designs at lower cost, but without sacrificing safety margins.

Almost all of the structures noted in the preceding paragraph were constructed using working stress design methods, that is, with a conventional safety factor. Beginning in 2012, foundations for major occupancies, including multi-family and commercial buildings in Winnipeg are required to be designed following Limit States Design methods. Bridges are designed following similar methods. The arrival of Limit States Design has led to a significant shift in the way local geotechnical practitioners design and view deep foundations, in particular those accustomed to working stress design methods. The introduction of Limit States Design, however, has fostered better communication between geotechnical and structural engineers: both now use similar design approaches and concepts to reach a common goal of achieving an adequate and consistent level of safety as well as minimizing damage and loss of function. The introduction of resistance factors that depend upon the quality of information has provided the impetus to carry out load testing to quantify foundation performance in terms of both the Ultimate Limit State and the Serviceability Limit State. The Author suggests that load testing will be seen as the best method to balance local knowledge, the level of risk, and the cost of foundations.

For as long as development encroaches on the city's waterways, riverbank stabilization will be an important aspect of geotechnical practice. In the last few decades, local understanding of riverbank stability has increased tremendously. This has largely been a result of the courageous efforts of local practitioners, educators and contractors to develop novel and more efficient stabilization techniques. This has not come without both risk and reward and the acceptance that an error or failure may occur along the way. There is no such thing as risk-free innovation. If creative thinking can be stifled by concerns such as professional liability, then it can also be nurtured by the application of practical experience.

One of the most challenging problems is often to differentiate alluvial soils from lacustrine soils and assign the appropriate shear strength properties. Even on an inside bend of one of the major rivers where alluvial soils would be expected, it may be difficult to determine the contact with lacustrine clays and straight or transitional channel sections (from an inside to an outside bend) could be either alluvial or lacustrine. Soil index properties can provide an indication of the depositional history and thus the expected soil type. The role of groundwater in slope stability is significant, in particular where slip surfaces extend to the till surface and shear strengths may be adversely affected by elevated piezometric levels. This condition is of the greatest importance during fall drawdown of the river and before aquifer pressures decline.

The appropriate shear strengths for modeling depend on whether the soil has been classified as alluvial or lacustrine, or more specifically, on its plasticity. There is a reasonably well defined trend towards decreasing friction angle with increasing plasticity, with all of the data from Winnipeg falling between the mean and lower bound curves derived from published data sets. "Large strain" values of c' = 5 kPa, ϕ'_{nc} = 14° are considered a reasonable lower bound approximation for lacustrine soils where groundwater gradients are incorporated in the model. The use of shear strengths of c' = 5 kPa, ϕ' = 17° yield similar results using an assumed phreatic surface. This is inconsistent with the observation that the strength envelope for Winnipeg clays is non-linear, in particular at low confining pressures where the true cohesive intercept c'=0 kPa. The inclusion of a small amount of cohesion however, yields stability analysis results more closely matching observed behaviour. Interestingly, the use of a bi-linear strength envelope in conjunction with near-surface soil suction yields similar results.

Failed portions of the bank are often assigned residual strengths which typically are back-calculated based on an assumption that the existing safety factor for the failed portion of the bank is unity (1.0). The back analysis typically assumes near- zero cohesion with ϕ' = 8-10° although larger friction angles are often calculated or measured for alluvial soils. The residual friction angle can also be measured in the laboratory using direct shear or ring shear tests and has been shown to vary with plasticity index. Careful examination of the soil profile should be made to detect the presence of lacustrine clay below the alluvial material as this layer may be weaker and hence dominate the calculated safety factor and geometry of the slip surface.

Keeping pace with the increased understanding of riverbank behaviour has been the development and refinement of slope stabilization techniques. From timber ribs to granular shear keys to rockfill columns, researchers and designers have pushed to develop and justify more economical methods of construction. Perhaps one of the greatest advancement has been the use of increased friction angles for rockfill material based on the results of laboratory, full-scale field testing, and the development of effective compaction methods. This alone has resulted in significant reductions in the size of the works which in turn reduces costs and risks during construction. Although currently, the cost of rockfill columns are about double that for an equivalent shear key, the risks of slope movement during construction are much lower, an important consideration if infrastructure may be impacted by movements.

For any of the stabilization works, compaction of rockfill is considered essential. Until the material has been compacted, large movements can be expected until significant shearing resistance is developed. For example, rockfill columns are often installed prior to compaction and significant movements may occur in the intervening time. It is important to recognize that not only is this movement non-recoverable, but any locked-in stresses developed are lost during compaction. If possible, rockfill material should be compacted as the columns are installed.

8.2 CONCLUDING COMMENTS

Unlike many other disciplines, geotechnical engineering involves the use of complex materials whose properties are highly variable and behaviour mechanisms often uncertain. Since we cannot simply specify the materials or geology we would like to work with, we must use what the natural environment has provided. Geotechnical practitioners must therefore make realistic assumptions during design, which ultimately will be tested against reality in the context of acceptable performance. While the ability to make realistic assumptions evolves from experience, some of the most valuable experience may be the consequence of making unrealistic assumptions. What better reason to seek the advice from an experienced practitioner?

In their designs, geotechnical engineers must often incorporate the physical reality of sub-surface conditions into a mathematical model to predict behaviour and performance. This solves problems on a purely theoretical basis. Although these methods provide a much greater ability to predict the behaviour of a soil, they are only as valuable as the state of reality on which they are based. Often, there are problems which are too complex or there is insufficient information, to justify the use of advanced modelling techniques. It is at an early stage in the design when an experienced geotechnical engineer can help conceptualize the model's outcome. This does not mean that the modeling inputs should simply be manipulated to match the expected outcome, but only that junior engineers can use this as an opportunity to apply their own intuition and explore the sensitivity of certain variables to develop a better understanding of the system.

In these circumstances, experience in conjunction with an understanding of the basic soil mechanics principals involved can play a valuable role in arriving at realistic solutions.

This is not to say that failures can be avoided entirely or that junior engineers should be discouraged from thinking further, only that the experience of others should be taken advantage of. This experience is timeless, and the lessons learned one hundred years ago are just as valuable today, perhaps even more-so if this experience is based on a failure. Even with the best modelling techniques, we must still imagine how a failure will occur in order to determine the level of safety. A word of caution though - it is wise to understand that design envelopes can only be pushed so far, and experience should be seen as benefiting, but not replacing, the often complex (but powerful) modern design methods at our disposal.

In preparing this thesis, the Author has come to appreciate the direct link between the fields of geotechnical engineering, geology and hydrogeology. Traditionally, the role of a geologist has been identifying geologic origin and processes over a large area, perhaps hundreds of square kilometres. Likewise, the traditional role of the hydrogeologist has been to study the distribution, flow and quality of groundwater, often on an extensive regional basis. However, the importance of geology and hydrogeology on smaller scale geotechnical projects cannot be overstated. An understanding of the basic principles, terms and properties of the variability and properties of local geology and geomorphology leads to more effective communication, identification of potential problems and the development of cost effective solutions to those problems. Local examples include the design and construction of rock-socketted caissons, which by their nature, directly relate to geology (in terms of the rock) and hydrogeology (in terms of groundwater inflow or quality).

Geotechnical practitioners must often strike a balance between a conservative design based on limited information and a less conservative design based on more extensive (and hence costly) information on soil and groundwater properties and behaviour. This balance is a basic outcome of the level of uncertainty in both material properties as well as expected performance. However, the link between the degree of uncertainty and design is not as straight-forward as simply assigning what is determined to be an appropriate safety factor. A high degree of uncertainty would in most cases be expected to produce an overly-safe but perhaps uneconomical or wasteful design. Conversely, a high degree of uncertainty can result in an unsafe design that may have much more severe consequences of failure. The introduction of Limit States Design has helped alleviate this quandary by assigning resistance factors to decrease the geotechnical resistance based on reliability theory taking into consideration the level of investigation of laboratory testing. At the same time, structural engineers apply similar factors to increase the anticipated loads based on reliability theory.

Design assumptions are the basis on which geotechnical analysis proceeds. However, even with the benefit of experience and judgement, it is wise to remember that the design is often a hypothesis based on an unproven theory or statement. While the design can generally never be proven to be absolutely correct, it can certainly be shown to be false through a failure. Local examples include the development of riverbank stabilization techniques on which the design and expected performance are based largely on an educated hypothesis, often made by on the basis of empiricism and precedent. While not all of these projects have been successful, the evolution of testing our design hypotheses has led to a much better understanding of the nature of the problem and the determination of economical solutions.

Previous chapters show the value of both past experience in the city and the adoption of modern understanding of ground water issues and numerical analysis. Evidence from this review of Winnipeg geotechnical practice supports the hypothesis proposed in Chapter 1, in particular the importance of junior engineers taking advantage of the past experience of early practitioners and senior engineers and using this experience as a guide to improve judgement and capacity for professional practice.

8.3 RECOMMENDATIONS FOR FUTURE WORK

One of the most significant changes in local geotechnical practice in recent years has been the use of limit states for foundation design rather than the traditional safety factor approach. While there is little doubt about the benefits of this design approach, and not withstanding that it is here to stay, several shortcomings in local practice are evident. The most significant of these is the assignment of serviceability limit states based on empirical data on foundation performance. What stands out in particular is the lack of load test data on cast-in-place concrete friction piles (for low to moderately loaded structures), driven precast concrete piles (moderate to heavily loaded structures), and rock-socketted caissons (heavily loaded structures). The author recognizes that considerable information is now available on driven steel piles through extensive PDA testing. This effort by a small number of individuals is valuable and should be acknowledged.

It would be helpful to carry out cast-in-place concrete friction pile load tests to measure shaft resistance and provide better measurement of the empirical adhesion coefficient (α) used in local practice for design of friction piles. These tests could be run using conventional load test procedures (reaction beam and jacking load), or an Osterberg cell (O-cell) testing method. It would be of value to also determine the end bearing contribution in cast-in-place concrete friction piles to confirm if its inclusion in the ultimate limit state is justified. It will also be important to measure the undrained shear strength profile along the length of the test pile, ideally with a comparison between unconfined compression and undrained unconsolidated (UU) tests in a triaxial cell. This will assist in determining if the unconfined compression test method yields lower results than would be measured using more advanced testing methods.

Load tests on precast concrete friction piles would be useful to confirm if higher nominal values can be justified. This work should include the assessment of pile performance for piles driven to practical refusal in till of varying consistencies. Static load tests in combination with PDA testing would be of considerable value.

Load tests (O-cell) have been carried out on two occasions on rock socketted caissons in the city although the results are unpublished. Given the costs associated with this foundation type, the cost of the testing could easily be recovered by a more economical foundation design. Of critical importance however, will be the ability to apply design guidelines for piles installed in a geological unit (limestone) with a wide range of competency and hence engineering properties important in design. It is therefore recommended that testing be carried out in a variety of bedrock conditions identified through detailed drilling and coring. Socket lengths should be varied to investigate the load sharing mechanisms between shaft resistance and end bearing.

The role of groundwater in geotechnical engineering design and construction is well documented. The recovery of groundwater levels over recent years is therefore of considerable interest to local practitioners as it may have a significant negative impact on deep foundations, excavations and riverbank stability. Further studies would help quantify the changes, notably in the downtown area and the vicinity of major waterways.

Mapping of riverbank soils would be of considerable value to local practitioners. This could be achieved by incorporating sub-surface data from site investigations into a data base linked with ground topography, ideally in a GIS platform. Such a collaborative effort would almost certainly require the support of the City of Winnipeg Waterways Section and local consultants. Intellectual property ownership may be a stumbling block but the Author is convinced the concept is worth exploring further. Further evaluation of the role of soil suction (negative pore water pressure) in riverbank stability may be warranted.

Further work is recommended to evaluate the physical properties of crushed limestone relative to the intended construction applications. This would include further studying the relationship between the Iowa Pore Index Test and soundness loss testing.

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