LIMIT STATES AND RELIABILITY-BASED DESIGN METHODS APPLIED TO THE BUOYANCY ASSESSMENT OF THE SHOAL LAKE AQUEDUCT

by

GILBERT G. ROBINSON

A Thesis
Submitted to the Faculty of Graduate Studies
in Partial Fulfilment of the Requirements for the Degree of

MASTER OF SCIENCE

Department of Civil and Geological Engineering
University of Manitoba
Winnipeg, Manitoba

0-612-76862-7

THE UNIVERSITY OF MANITOBA

FACULTY OF GRADUATE STUDIES
*****
COPYRIGHT PERMISSION PAGE

LIMIT STATES AND RELIABILITY-BASED DESIGN METHODS APPLIED TO THE BUOYANCY ASSESSMENT OF THE SHOAL LAKE AQUEDUCT

BY

GILBERT G. ROBINSON

A Thesis/Practicum submitted to the Faculty of Graduate Studies of The University of Manitoba in partial fulfillment of the requirements of the degree of

Master of Science

LIABILITY DISCLAIMER

This thesis document has been prepared in partial fulfillment of the requirements for the Degree of Master of Science at the University of Manitoba. The thesis is based on work completed by UMA Engineering Ltd. and CH2M_Gore and Storrie Ltd. for the City of Winnipeg, Water and Waste Department. It contains additional material that has been specifically prepared by the author to satisfy the requirements for the degree program. The ideas and views presented in this thesis are entirely those of the author and not UMA Engineering Ltd. or CH2M_Gore and Storrie Ltd. The purpose of this disclaimer is to relieve UMA Engineering Ltd. and CH2M_Gore and Storrie Ltd. of any liability that may be associated with the contents of this thesis.

ABSTRACT

An actual design example was used to demonstrate how limit states design methods and engineering judgement can be applied to solve a non-codified engineering problem. The work was focussed on the assessment of potential buoyancy during shutdowns of the eighty-three year old Shoal Lake Aqueduct that is the sole source of potable water for Winnipeg, Manitoba. Four separate buoyancy analyses of the Shoal Lake Aqueduct were completed. Three of the analyses were conducted using limit states and reliability-based design methods. For comparative reasons a fourth analysis was completed using a traditional working stress design (WSD) method. A buoyancy model was developed and used in these analyses. The first buoyancy analysis was completed using partial safety factors developed for the Ontario Highway Bridge Design Code. Because these partial safety factors were developed for use with the design code a second analysis was completed using project-specific partial safety factors. These partial safety factors were determined using Second Moment reliability techniques and measured data for the uncertainties in the buoyancy model. A third buoyancy analysis was completed using Monte Carlo simulation techniques. A fourth buoyancy analysis was completed using WSD methods to demonstrate the potential variability in the level of safety. Engineering judgement was required to develop the buoyancy model, to interpret the data obtained for each of the parameters and to provide meaningful design values for those parameters which could not be measured.

The results of the buoyancy analyses completed using limit states design and reliability-based methods were similar. Because the partial safety factors from the Ontario Highway Bridge Design Code were not based on the measured variability of different parameters, the potential for deviation from a target level of safety is significant. The target level of safety provided using project-specific partial safety factors and Monte Carlo simulation is more reliable because the results reflect the measured variability of the parameters. The target level of safety using WSD methods is not directly quantifiable. The results of this thesis show that the selection of a single factor of safety has a very significant influence on the target level of safety, that is it does not give uniform reliability.

ACKNOWLEDGEMENTS

I would like to acknowledge the support of my advisor, Dr. James Graham. He encouraged me to rejoin the Master of Science program and his tireless efforts were instrumental in getting this thesis done before his retirement at the end of June 2002. His dedication to his students is remarkable and appreciated by all who have worked with Dr. Graham. Enjoy your retirement Dr. Graham, take pride in knowing how many students you have helped over the years.

I would also like to thank the three partners in the Shoal Lake Aqueduct Condition Assessment and Rehabilitation Program: the City of Winnipeg, Water and Department, CH2M_Gore and Storrie Ltd. and UMA Engineering Ltd. In particular, I would like to thank Ron Sorokowski from the City of Winnipeg; Chris Macey, Ken Skaftfeld and Tom Wingrove from UMA Engineering Ltd. These organizations and people gave me full access to all the resources I needed to finish this project.

Thanks also go out to Dennis Becker of Golder Associates Ltd. who provided some input and reference material over the course of writing this thesis and to Dr. Dave Rogowsky from the University of Alberta who took time out of his busy schedule to come to Winnipeg to be part of the thesis examining committee.

Finally, I would like to express my heart-felt appreciation and love to my family who provided encouragement, understanding and support over the last year while I was writing this thesis document.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
ADD	average daily demand
BR1	Branch 1 Aqueduct
BR2	Branch 2 Aqueduct
CHBDC	Canadian Highway Bridge Design Code
GFS	global factor of safety
GWWD	Greater Winnipeg Water District
LF	load factor
LRFD	load and resistance factor design
LSD	limit states design
MLD	millions of litres per day
NBCC	National Building Code of Canada
OHBDC	Ontario Highway Bridge Design Code
PDF	probability density function
P_f	probability of failure
P-S	project-specific
PSF	partial safety factor
RBD	reliability-based design
RF	resistance factor
RR	resisting ratio
SLA	Shoal Lake Aqueduct
SLA_CARP	Shoal Lake Aqueduct Condition Assessment and Rehabilitation Program
SLS	Serviceability Limit State
ULS	Ultimate Limit State
WSD	working stress design

1.0 INTRODUCTION

In the mid-1970's, limit states design (LSD) methods were introduced to Canadian structural engineers as an alternative to working stress design (WSD) methods. (A list of abbreviations used in this thesis is given on page xi following the Table of Contents.) Since this time, LSD methods have become the general state of practice for structural design, with their objective being to provide a more quantifiable and consistent level of safety. In contrast, geotechnical engineers have been slow to adopt LSD methods to replace WSD methods. The slow progress within the geotechnical community can be attributed, at least in part, to the years of experience and level of comfort using WSD and the high degree of uncertainty associated with many geotechnical parameters.

Limit states are defined as conditions under which a structure or its component members no longer perform their intended functions (Becker, 1996a). The use of LSD methods is facilitated with the use of design codes. The objective of design codes is to ensure a minimum level of technical quality and a minimum or specified level of safety. The level of safety or reliability of a system can be defined in terms of the probability of failure. In general, this is achieved through the use of multiple load and resistance factors, specified design values for material properties and specified load combinations. The purpose of load and resistance factors, or partial safety factors (PSF), is to account for uncertainties, for example, in the measurement of material properties, uncertainties in analytical models, and uncertainties in loads that the structure is required to sustain. The PSFs can be determined directly using probabilistic or reliability methods which make use of large databases of measured values, or they can be determined by direct calibration with WSD methods.

In contrast, WSD only uses one global factor of safety (GFS) to account for all uncertainties. This GFS is almost entirely based on engineering judgement and experience and it would be extremely difficult to estimate a level of safety (or a probability of failure) for any value of the GFS.

1.1 STATEMENT OF PROBLEMS

There are two general problems. First, LSD methods are not compatible with WSD methods. Two, many classes of engineering problems are not covered by design codes based on LSD, particularly engineering problems in the general area known as geotechnical engineering.

The first problem is best explained by example. Assume a geotechnical engineer, using WSD, provides foundation recommendations for a spread footing to a structural engineer who will design a structure using LSD methods. The foundation recommendations are based on Terzaghi's bearing capacity equation, which is a strength equation. In traditional geotechnical practice, the ultimate bearing capacity calculated from this equation is reduced using a GFS ranging from 2.5 to 3. These relatively large GFSs are selected to provide not only a safety margin against catastrophic failure, but also, in a notional way, to restrict bearing pressures and limit settlements. In this example, the single GFS provides protection against exceeding both an Ultimate Limit State (ULS) and a Serviceability Limit State (SLS). The resulting "allowable bearing capacity', is the value provided to the structural engineer. Difficulties can develop when a structural engineer tries to evaluate ULSs using a value that is intended to limit settlements, which is considered to be an SLS. Obviously the two systems are not compatible and there is much potential for confusion and human error. In addition, the level of safety associated with the foundation recommendations may not be comparable with the level of safety for the structure itself. The problem arises because LSD methods provide separate evaluations of ULSs and SLSs, which are related to strength and deformation, respectively. In contrast, GFS methods combine these quite separate states through a single 'confidence' factor that is purely empirical.

The second problem, and the focus of this thesis, is that many classes of engineering problems are simply not covered by current design codes based on LSD. This is particularly true for existing infrastructure and geotechnical engineering problems. In these cases, engineers may feel that WSD methods are more attractive due to their relative simplicity. The level of safety provided by WSD would be unknown. Engineers might therefore prefer to use a Limit States approach. However, if no design code has been prepared, it may indicate there is a lack of data that can be used to derive PSFs for many of the parameters used in geotechnical design.

1.2 LIMIT STATES DESIGN IN GEOTECHNICAL ENGINEERING

In 1980, an agreement was reached to draft a geotechnical design code for use across Europe (Ovesen, 1981). Some countries, such as Denmark, had been using design codes with prescribed PSFs for quite a few years. This generated quite a bit of interest from European engineers using WSD methods. Many questions were raised about the pitfalls that might be encountered using prescribed PSFs or attempting to account for the variability of geologic uncertainties using statistics and probability theory. Many felt that the use of LSD and prescribed PSFs would remove the engineer from the design process and there would be no room for engineering experience or judgement. From about 1981 to the early 1990's, there seems to be a distinct gap in published literature on the use of LSD in geotechnical engineering. Since that time a considerable amount of work has been completed and published in conference proceedings (International Limit States Design Symposium, 1993; Uncertainty in the Geologic Environment, 1996) and engineering journals such as the Canadian Geotechnical Journal and the ASCE Journal of Geotechnical Engineering. In addition, three Canadian limit states design codes now require the use of LSD for the geotechnical component of design. These codes include the Ontario Highway Bridge Design Code (OHBDC; Ministry of Transportation of Ontario, 1991), the National Building Code of Canada (NBCC; National Research Council of Canada,1995) and the Canadian Highway Bridge Design Code (CHBDC; Canadian Standards Association, 2000). Limit States Design procedures will be introduced in the next edition of the Canadian Foundation Engineering Manual published by the Canadian Geotechnical Society. The new edition is expected in 2003-2004.

The use of LSD methods by Canadian geotechnical engineers has been initiated through the design codes mentioned above. However, many of the problems faced by geotechnical engineers do not have design codes to guide the engineer. How can engineers apply LSD methods to non-codified problems? Can engineers use prescribed PSFs from other codes? What risk would be associated with this? Do engineers need to use PSFs or can reliability theory be used to help evaluate the level of safety? How much understanding of probability and reliability theory is required? Can engineering judgement be used in conjunction with LSD methods?

These issues are addressed in this thesis by way of an actual design example, which involved assessment of possible buoyancy of the Shoal Lake Aqueduct in eastern Manitoba. The eighty-three year old Shoal Lake Aqueduct is the sole water supply for Winnipeg, Manitoba. It was constructed between 1915 and 1919 and has been in almost continuous use since that time. Only in recent years has the aqueduct been shutdown to facilitate much needed maintenance repairs. During these shutdowns, there is a possibility that the aqueduct could become buoyant in areas where the aqueduct is submerged and the backfill material is mainly comprised of light-weight organic soils. No design code exists to evaluate the potential for buoyancy of a structure like the aqueduct. One of the biggest advantages the engineer has is that the aqueduct has already been constructed. This permits direct measurement for many of the parameters or uncertainties involved in assessing buoyancy potential. Direct measurement was necessary because there is no database available that can be used to determine appropriate design parameters or PSFs that can be used in design.

This LSD example is a case study of a unique and rare engineering problem involving a very critical component of civil engineering infrastructure. There is no design code directly applicable to buoyancy assessment and there is minimal engineering experience. The amount of field information that was required to solve this problem is not ordinarily available to students and therefore the work in this thesis represents a unique opportunity to show how LSD methods can be applied outside of design codes. This work is not simply the application of probabilistic and reliability methods. It involves a significant component of engineering judgement to develop the buoyancy model and to interpret the data used in the model.

HYPOTHESIS:

A combination of limit states design (including reliability-based design methods) and good engineering judgement can provide technically sound, economic solutions to many classes of engineering problems, particularly problems involving remediation of existing infrastructure.

1.3 OBJECTIVES

The objectives to be met in this thesis include the following.

1. To show that LSD methods can be successfully applied to non-codified engineering problems.
2. To show why PSFs from non-related design codes may not be applicable.
3. To show how project-specific PSFs can be developed.
4. To show how reliability-based design methods can be used as an alternative to the PSF method and to check the results of designs completed with project-specific PSFs.
5. To demonstrate the inherent uncertainty in using WSD methods.
6. To demonstrate the importance of engineering judgement in design.

1.4 ORGANIZATION OF THESIS

Following this introduction, Chapter 2 presents a general overview of LSD. The chapter begins by outlining the various uncertainties and required level of safety that need to be identified before design begins. For comparative purposes, a brief discussion on WSD has been included. The section on LSD provides a definition and history of LSD, an overview of the various LSD methods that can be used in design and a discussion of the concerns raised by engineers regarding the use of LSD methods in geotechnical engineering. A general approach to applying LSD methods to non-codified engineering problems is included at the end of the chapter.

Chapter 3 provides an overall summary of the Shoal Lake Aqueduct. The chapter highlights the engineering challenges faced in the early 1900's by the engineers during the design and construction of the aqueduct. The operating history of the aqueduct has been included to show how the operating requirements have changed over the years and how these relate to buoyancy potential of the aqueduct. The chapter concludes by introducing the Shoal Lake Aqueduct Condition Assessment and Rehabilitation Program that was initiated by the City of Winnipeg, Water and Waste Department in the early 1990's.

Chapter 4 provides specific information regarding the Buoyancy Assessment Program for the aqueduct. The concern regarding buoyancy of the aqueduct is discussed along with the buoyancy concept for the aqueduct. The field-work that was completed over the course of the Buoyancy Assessment Program is reviewed and pertinent data presented. The chapter concludes with a general overview of the various options that were evaluated in selecting a preferred remediation plan to reduce the risk of buoyancy of the aqueduct.

Chapter 5 provides details of four different buoyancy analyses that were completed using both LSD and WSD methods. The discussion starts with a preliminary buoyancy analysis used to evaluate the importance of the different parameters used in the analysis. The various LSD methods and buoyancy models used are compared with each other and also with the WSD results. The required use of engineering judgement is evident in the discussion. Chapter 6 details the conclusions that can be drawn from the work completed in this thesis and provides recommendations for further work.

This thesis document is based on project work completed by the author during his employment as a geotechnical engineer by UMA Engineering Ltd. The project was completed in partnership with CH2M_Gore and Storrie Ltd. of Toronto, Ontario and the City of Winnipeg, Water and Waste Department. For academic purposes, the thesis contains a considerable amount of additional original work undertaken by the author to broaden the scope of the original reports provided to the City of Winnipeg and to pursue in greater detail the technical value of applying LSD methods to this class of civil engineering problem.

2.0 LIMIT STATES AND RELIABILITY-BASED DESIGN

2.1 INTRODUCTION

In North America, limit states design (LSD) has been used in structural design since the mid-1970's. The alternative to LSD is working stress design (WSD). It was first introduced in the early 1800's (certainly in North America) and has been used almost exclusively by geotechnical engineers. In WSD, all uncertainty is incorporated into design by a single global factor of safety (GFS). The value chosen for the GFS is typically based on engineering experience. Its value is generally different for different classes of engineering problems. Global factors of safety do not provide a quantifiable measure of the level of safety (reliability)- they simply give engineers a general sense of the level of safety of the structure. In contrast, LSD requires that all limit states, ultimate and serviceability, be identified and checked. A quantifiable level of safety can be incorporated into the design through the use of probabilistic and reliability methods.

Limit states design methods for geotechnical design were first introduced in Europe in the 1950's and Danish engineers have been using their own LSD code since the mid-1960's. The first LSD-based code for foundations was not introduced in Canada until 1983 when it was first included in the Ontario Highway Bridge Design Code (Code (OHBDC; Ministry of Transportation of Ontario, 1983). Limit state design procedures have been receiving increased attention, both good and bad, over the last two decades. Many engineers feel that the role of engineers in the design process will be significantly reduced with the use of design codes that are based on limit states and use prescribed partial safety factors. Others feel that more judgement and understanding of the different failure modes will be required. Through the use of an actual design example, this thesis shows how limit states and reliability-based design methods can be adapted to a unique engineering problem for which limited past experience exists. Because this thesis deals with a soil-structure interaction problem, the following discussion implicitly includes both geotechnical and structural aspects.

2.2 UNCERTAINTIES AND SAFETY IN DESIGN

2.2.1 Uncertainties in design

Uncertainty is inherent in all engineering work due to natural variability in ground conditions and construction materials, inaccuracies in design methods, variability in loads and resistances and errors in testing procedures, amongst others. In order to minimize the probability of failure of an engineered structure, these uncertainties must be accounted for. Traditionally, this was done using a GFS approach. More recently, partial safety factors (PSF) have been developed using probability theory, reliability methods, and direct calibration with WSD methods.

Uncertainties in engineering have been well discussed by others, for example Becker 1996a, MacGregor 1975, Morgenstern 1995, Phoon and Kulhawy 1999, the ASCE Proceedings from Uncertainty in the Geologic Environment: From Theory to Practice, 1996; and the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, 1993. The following discussion is based on the findings from the references listed above.

There are two broad categories of uncertainties in engineering, namely objective uncertainties and subjective uncertainties. There are four main classes of objective uncertainties namely, ground conditions, measurement of material properties, model uncertainties and load uncertainties. Subjective uncertainties include human errors, gross errors, workmanship and engineering judgement. Objective uncertainties are more quantifiable than subjective uncertainties and can therefore be more easily accounted for during design. Subjective uncertainties are much less quantifiable and therefore hard to, if not impossible, to account for in design. Some subjective uncertainties can be accounted for by quality control during design and construction (Meyerhof, 1984). An example of quality control during construction is measuring the density of compacted materials to ensure that a minimum density is achieved during construction.

In general, there are two general types of construction materials, structural and geotechnical. Manufactured structural materials such as steel and concrete are much more uniform than geotechnical materials and therefore have a much lower degree of uncertainty. This lower degree of uncertainty is mainly due to the quality control procedures used during the manufacturing process for steel and concrete. In contrast, uncertainty is inherent in geotechnical materials due to the complex geologic processes under which the materials were deposited.

Uncertainty in geotechnical materials can be divided into two groups: aleatory uncertainty and epistemic uncertainty (Baecher and Christian, 2000). Aleatory uncertainty is the natural randomness of a property. It cannot be reduced or eliminated. Epistemic uncertainty is due to a lack of knowledge and can be improved by collecting more information. For example, measuring the undrained shear strength of clay at only two locations would not provide a meaningful range of shear strength values. As more measurements are made the uncertainty in the range of shear strength reduces. Unfortunately, instrument errors and procedural errors introduce additional uncertainty with any form of measurement.

Bolton (1981) believes that the overwhelming majority of uncertainties in geotechnical design are of the geotechnical system, rather than of its parameters, and that the automatic application of statistical methods of any sort is fraught with danger and paradox. However, one must consider that engineering experience combined with statistics may allow engineers to better understand the variability of different parameters.

Model uncertainties are usually greater for geotechnical applications than structural applications because the behaviour of geotechnical materials is not as well defined as structural materials, particularly in the case of soil-structure interaction models. Model uncertainty arises due to the idealizations and assumptions that have to be made in the formulation of a physical problem. Model uncertainty can be sometimes accounted for by a bias factor, which is determined by comparing the predicted performance of a structure to the actual performance. Determining actual performance requires full-scale field-testing such as pile load tests.

There are two general load types, dead loads and live loads. Load uncertainties are less of a concern for dead loads than live loads because dead loads can be calculated more accurately than any other loads except possibly fluid loads (MacGregor, 1975). Live loads vary considerably with time and from structure to structure due to change in use and climate, among others.

2.2.2 Level of safety

In designing any type of structure, the various uncertainties described in the previous section must be accounted for if an adequate level of safety is to be provided against failure. This level of safety depends on the class and importance of the structure and the consequence of failure. The level of safety, or probability of failure, also needs to be comparable to the risks that people are willing to accept in specific situations or from natural events or from natural or manmade works. (Becker, 1996a)

MacGregor (1975) identified a number of subjective values that should be considered when attempting to determine the consequences of failure. These values include cost of replacement, potential loss of life, costs to society in lost time, lost revenue or indirect loss of life or property due to failure, importance of different components in the structure, and the type of failure, warning of failure and existence of alternative load paths.

MacGregor (1975) also went on to summarize some statistical data showing the yearly death rate per person per year for various activities. The results were grouped into levels of acceptable risk which are summarized as follows:

- Avoidable risks connected with daring people 1/1000 per year
- Avoidable risks connected with careful people 1/10,000 per year
- Unavoidable risks 1/20,000 per year

These results suggest the probability of failure of a structure through collapse that results in one death should be about 1/20,000. If more deaths may result an even lower probability of failure should be used. Based on these results and results from a German study on roof snow loads, MacGregor (1975) then suggested, for the work he undertook, that the probability of failure should not be less than about 1/100,000 per year. MacGregor's work was based on new construction where the cost associated with extra safety is small.

More recently, the desired level of safety (risk) for various types of structures (Figure 2-1) has largely been based on observed risks for certain natural events and engineering projects designed in keeping with current engineering practice

(Becker, 1996a). The results presented in Figure 2-1 indicate that the risk associated with engineering works is less than the generally accepted risk, which is in turn, less than the risk associated with natural events such as hurricanes and earthquakes. Becker (1996b) states that the National Building Code of Canada (NBCC) for structural design was calibrated using a target probability of failure of 10^{-4} in 30 years for ductile behaviour with a normal consequence of failure and a target probability of failure of 10^{-5} in 30 years if the consequence of failure is severe or the failure is likely to occur in a brittle manner. A target probability of failure of 10^{-4} in 30 years was used for code calibration for foundations in the NBCC (1995).

There are numerous ways in which safety has been incorporated into engineering design. Traditionally a single global factor of safety has been used in conjunction with WSD. More recently, partial safety factors based on probability and reliability theory and also direct calibration with WSD have been used with LSD. Reliability-based design methods are also used, but to a much lesser extent. These methods will be discussed in more detail in Section 2.4.4. It is important to note that methods of safety analysis are not designed to produce true estimates of safety but should be seen as aids in the process of controlling safety (Oliphant, 1993).

In referencing a report on safety in structural codes, Bolton (1981) implies that ninety percent of failures occur because the design calculations used were irrelevant to the structure that was actually created. That is, ninety percent of structural failures are incapable of prevention by classical reliability theory. Bolton then questions the percentage of geotechnical failures for which the theory will be useful. It would be valuable, and interesting, to know the percentage of failures based on the total number of structures built. This will likely be a relatively small percentage, which would indicate that the design methods used have performed their intended function of ensuring a sufficient level of safety. What Bolton fails to mention is that no design method can be expected to cover situations such as misinterpretation of site conditions, incorrect assessment of soil properties, or use of inappropriate models. These gross errors can only be handled by ensuring proper professional competence and conduct on the part of the engineer (Phoon et. al., 1993).

2.3 WORKING STRESS DESIGN

2.3.1 Background

Working stress design methods have been used in civil engineering since the early 1800's and are still used today, predominantly by geotechnical engineers. The objective of WSD is to ensure that the stresses generated by service loads are less than the allowable stresses of the structure by an empirical factor or GFS chosen on the basis of past experience. This GFS accounts for the various uncertainties in design that were discussed in the previous section. Working stress design is essentially a deterministic approach that assumes the loads and resistances are well defined.

The GFS can be applied to the resistance or the applied load (Equation 2-1) depending on which term has the most uncertainty, traditionally the GFS is applied to the resistance. The magnitude of the GFS varies for each common type of design and varies over a range of values that are largely based on past experience. In general, higher values in the chosen range are used when there is more uncertainty in the variables and/or there is a higher consequence of failure. A limiting condition exists when the GFS = 1, which implies that the loads and resistances are just equal and any further reduction in resistance and/or load increase will result in failure.

[2-1] $$GFS = \frac{\text{Ultimate Resistance}}{\text{Applied Load}}$$

The load and resistance terms in Equation 2-1 may include forces, moments, temperature changes, water pressure changes, and chemistry changes among others. In many situations, the geotechnical GFS is selected to control deformations such as settlements and slope movements.

2.3.2 Limitations

Working stress design is a simple and straightforward method of accounting for uncertainties in design and has been used somewhat successfully over the years. However, Becker (1996a) pointed out some major drawbacks in WSD:

- WSD is not compatible with current LSD methods used in structural engineering,
- there is no rational or consistent way to define the level of safety or probability of failure through the use of GFSs,
- the expected level of safety provided through the use of a particular GFS is based almost entirely on experience,
- the level of safety between similar classes of problems is highly variable even for a constant GFS because engineers tend to use different design methods and select different design values for strength,
- the engineer is not forced to consider how a structure will perform under ultimate and serviceability states,
- The use of a single GFS does not separate or distinguish between the various sources of uncertainty,
- There is no direct (inverse) relationship between GFS and probability of failure.

Despite the shortcomings of WSD and single global factors of safety, knowledge gained over the years provides a valuable source of information that can be used to calibrate LSD methods so that geotechnical engineers can begin to use a more rational design method that facilitates determining the level of safety. Many engineers are comfortable with the WSD method and feel that the LSD process is just a more complicated means of achieving similar results that only reduce or eliminate the role of the engineer (Day 1997). As engineers, we should be looking for better ways to design and build structures with a more consistent level of safety.

2.4 LIMIT STATES DESIGN

2.4.1 Definition of Limit States Design

Limit states are defined as conditions under which a structure or its component members no longer perform their intended functions (Becker, 1996a), these conditions are identified and considered in the design process. The term LSD does not by itself, imply how uncertainties are to be accounted for during design. However, the term LSD is commonly associated with the use of probabilistic methods.

There are two general classes of limit states, ultimate and serviceability. Ultimate limit states (ULS) pertain to safety and failures such as structural collapse. Serviceability limit states (SLS) pertain to conditions that affect the functional use of a structure for its intended purpose. All potential limit states must be identified for each structure because the ULSs and SLSs vary with the type of structure and the intended use. Some examples of ULSs include the formation of a plastic hinge, buckling, loss of static equilibrium (buoyancy), overturning, sliding and ultimate bearing capacity. Some examples of SLSs include cracking, deformations, excessive vibrations, localized damage, excessive total and differential settlements. The occurrence of an ULS can result in loss of life and/or excessive financial costs. Structures are therefore designed with an acceptably low probability of occurrence of ULSs. The low probability of occurrence is achieved by ensuring that:

[2-2] Factored resistance $\geq$ Factored load effects

The resistance and load effects include factors such as force, moments, temperature changes, water pressure changes, and chemistry changes among others.

Load and resistance factoring can be achieved in a number of different ways that are discussed in more detail in Section 2.4.4. The consequences due to the occurrence of a SLS are much less severe, and therefore a higher probability of occurrence is generally acceptable. In the case of SLS the objective is to ensure that:

[2-3] Deformations $\leq$ Tolerable deformation

Tolerable deformation is that required for a structure to remain in service. Serviceability limit states are commonly checked using unfactored loads and resistances.

2.4.2 Historical Development of Limit States Design

The development of LSD in geotechnical engineering has been well discussed by others including Meyerhof (1995), Becker (1996a) and Ovesen (1981). In Canada, the current use of LSD is strongly associated with structural engineering largely because it has been used almost exclusively since the mid-1970's when it was included in the National Building Code of Canada (NBCC; National Research Council of Canada,1975) as an alternative to WSD for the structural design of buildings. However, the roots of LSD in geotechnical, and structural design, extend back as far as the 17th century (Hooke, Newton, Euler), 18th century (Coulomb) and 19th century (Rankine). The use of factors of safety were introduced in the 18th century by Belidor (1729) and Coulomb (1773); and became widely used in Europe and North America in the first half of the 20th century.

In the 20th century more insight was brought to the geotechnical community by the work of Terzaghi (1943), Taylor (1948) and Brinch-Hansen (1953, 1956). In 1943, Terzaghi identified two principal groups of problems, stability and elasticity. Stability problems, a form of ULS, consider the stress conditions just at failure (state of plastic equilibrium) without consideration of strain. Elasticity problems, a form of SLS, involve soil deformation (strains) without consideration of stress conditions for failure. In 1948, Taylor introduced the concept of separate or partial safety factors on the shear strength parameters c' and tan ϕ' used in slope stability work. In the mid-1950's Brinch Hansen proposed partial factors for loads, shear strength parameters and pile capacities for the ULS design of earth retaining structures and foundations. The first geotechnical design codes based on limit states were introduced in the mid-1960's by Denmark and followed by Romania in 1969. Other countries have since developed geotechnical design codes, including the USA, Czech Republic, former USSR, former German Democratic Republic, former Federal Republic of Germany, France and others. In Canada, it was not until 1983 when the OHBDC (1983) introduced LSD for foundations.

2.4.3 Current Status of Limit States Design In Geotechnical Engineering

There are a few countries, Denmark for example, that have successfully used geotechnical design codes based on limit states design principles for many years. There are however, a significant number of countries, including Canada, that have not fully embraced this design approach even though it is sometimes required by code for design of structures, such as bridges. Two general ways of incorporating limit states into geotechnical design have developed, a factored strength approach (Europe) and a factored overall resistance approach (North America). These will be discussed in Section 2.4.4.

In the early 1980's, the global geotechnical community was introduced to the LSD design concept. A significant number of concerns, which are discussed in Section 2.4.5, were raised at this time by, for example, Boden 1981; Bolton 1981, 1993; Semple 1981, Simpson et al. 1981, Fleming 1989 and Day 1997. These initial reactions resulted in a significant effort by the geotechnical community to find ways to incorporate LSD into geotechnical practice and account for various uncertainties such as that in the geologic environment. In 1981, the European's started the development of a geotechnical design code known as Eurocode 7 (CEN 1992), which is scheduled to be released around 2003. In North America, LSD was incorporated into bridge design codes such as the American Association of State Highway and Transportation Officials (AASHTO; 1994) and the OHBDC (1983) which has recently been replaced with the CHBDC(2000). In Canada, the LSD approach is also included in the NBCC (1995, 1996) and is also used for the design of offshore structures. Currently, the majority of geotechnical work completed in Canada is done using WSD. This leads to confusion when foundation recommendations based on WSD methods are provided to structural engineers who use LSD and must take account of both stability concerns (ULS) and settlements (SLS) in designing foundations. No codes currently exist in Canada for earth dams, embankments or slope stability. It appears likely that the forthcoming European Code (CEN 1992) will address embankments and slope stability but not dams and dykes (Orr and Farrell, 2000).

In the mid-1970's, the structural engineering community in Canada made a gradual change from WSD to LSD. This was accomplished in the NBCC (1975) by allowing designers to use either WSD or LSD. A gradual transition from WSD to LSD is also needed for the geotechnical community to help practising engineers understand and become familiar with LSD concepts. A large database and wealth of experience have been gained over the years of using WSD and these can be used in calibrating LSD codes. The objective of LSD methods is to provide a rational design approach with a more consistent and quantifiable level of safety. To meet these objectives we must allow engineers to utilize their experience and exercise their judgement but we must also bring a level of consistency to site characterization and selection of design values for strength and resistance. A site characterization manual, that will complement the Canadian Foundation Engineering Manual (3rd Edition), is presently being prepared by the Canadian Geotechnical Society and will to help achieve these objectives. Because of its better logic and ability to improve communications with other engineers, it is important to begin educating geotechnical engineers about LSD and to include LSD in the geotechnical education programs at all universities.

2.4.4 Limit State Design Methods

Many engineers associate LSD directly with probabilistic methods. Although the uncertainties in LSD may be dealt with using partial safety factors, which have been calculated using probabilistic and reliability methods, the main LSD concept is to identify and separately consider each potential limit state and ensure that the occurrence of each applicable limit state satisfies the design criteria, as indicated in Equations 2-2 and 2-3. Uncertainties in design in either the resistance term or the load effects term in Equation 2-2 can be accounted for using a variety of methods that are discussed below.

Two general approaches have emerged to deal with uncertainties on the resistance side of the general ULS equation (Equation 2-2), namely, the factored strength approach and the factored resistance approach. The European engineering community has developed the factored strength approach, which accounts for uncertainties by factoring down the strength parameters c' and tan ϕ' with the corresponding partial safety factors f_c and f_ϕ. These reduced strength factors are subsequently used to directly calculate a factored resistance. In North America, the factored resistance approach has been developed in which unfactored strength parameters are used to determine the overall resistance and this is subsequently reduced using a single partial safety factor, ϕ. In both approaches, the uncertainties associated with load effects are dealt with through the use of one or more load factors (LF) applied separately to each of the components of loading.

The factored strength approach has been used successfully in Denmark since the mid-1950's. Despite the fact that the approach implies that all the uncertainty is associated with the strength parameters, these factors have only undergone slight adjustments over the years to provide the required performance (Meyerhof, 1995). Other sources of uncertainty exist, as discussed in Section 2.2, including model uncertainty, measurement of material properties and uncertainties associated with ground conditions. Within the factored strength framework, these sources of uncertainty would require the use of separate partial factors. Eurocode –7, (CEN 1992) states that all factors or variables that influence soil strength, such as rate effects, stress path, stress-strain behaviour, and the inaccuracy in the design model are to be considered in the determination and selection of the characteristic strength values for c' and ϕ'. The characteristic strength is defined in Eurocode 7 as "a cautious estimate of the value affecting the occurrence of the limit state." However, no instructions or substantial guidance are provided on how this should be done. (Bengtsson et al. 1993)

In Canada, the factored resistance approach has been used in structural engineering since about 1975 and since the early 1990's to a much more limited extent in geotechnical engineering OHBDC (1991). Because only one resistance factor is used it is implied that it accounts for all uncertainties on the resistance side of Equation 2-2. The resistance factor can be determined to account for the combined effect of more than one uncertainty (MacGregor, 1975). The factored resistance approach has the added advantage that it can be used with empirical design methods, whereas the factored strength approach requires the use of a performance factor that serves the same basic purpose as a resistance factor. Calibration of the factored resistance with existing WSD methods appears to be more straightforward than calibration of the factored strength approach due to the reduced number of partial factors involved. The NBCC (1995) for foundations was calibrated with WSD methods to provide similar results (Becker 1996a).

At least six LSD methods have been proposed to account for the uncertainties in design. Becker (1996a) describes five of the methods including the load factor method, load and resistance factor design, LSD using worst credible values, LSD using extreme values, and reliability-based design. Day (1998) briefly describes the use of partial factors in design. These will be described in turn in the following paragraphs.

Load factor method

The load factor method does not use resistance factors, only LFs. This method is suitable only in cases when there is little uncertainty in the resistance. This is typically not the case for geotechnical structures and therefore will not be discussed further.

Load and resistance factor design method

The load and resistance factor design (LRFD) method is an extension of the load factor method but accounts for uncertainties associated with resistance using a single resistance factor. The load effects are determined using multiple LFs. Due to differing degrees and sources of uncertainty, uncertainties associated with the resistance and load effects are not related and must therefore be accounted for separately. Some examples of design codes in North America that use LRFD include OHBDC 1993, CHBDC 2000, NBCC 1995 and 1996 and AASHTO.

The LRFD approach provides structural safety by underestimating its resistance and overestimating the load effects to provide a factored resistance that is greater than or equal to the factored load effects. The level of safety is defined in terms of probability of failure, and the design is based on some acceptable level of risk or probability of failure. Figure 2-2 provides some examples of the variability of loads and resistances relative to each other, the shaded areas in this figure indicate the load and resistance combinations that could lead to failure.

The general LRFD equation is:

[2-4] $$\phi R_n \geq \Sigma \alpha_i S_{ni}$$

where

ϕR_n - factored resistance;
ϕ - resistance factor;
R_n - nominal resistance determined from engineering analysis;
$\Sigma \alpha_i S_{ni}$ - summation of the factored overall load effects;
α_I - load factor corresponding to a particular load, $S_{ni;}$
S_{ni} - a specified load component of the overall load effects;
$_i$ - represents various types of loads such as dead load, live load etc.

The left-hand side of Equation 2-4 is the factored resistance, which must be greater than or equal to the right-hand side, which represents the summation of the factored load effects for a given loading condition. The resistance factor, ϕ is less than 1.0 and accounts for variabilities in geotechnical parameters and model uncertainties. Resistance factors commonly range from 0.3 to 0.9. The load factors, α_i account for uncertainties in loads and their probability of occurrence. They can range from 0.85 to 1.3 for dead loads and from 1.5 to 2.0 for live and environmental loads. Dead load factors less than unity apply to situations where the dead loads produce a stabilizing effect, for example the resistance against overturning of a retaining wall. The main advantage of LRFD over WSD is that it can provide a more consistent and uniform level of safety for all load combinations, and different types of materials, structures and foundations (Becker, 1996a). For example, consider two structures or footings where one carries predominantly dead load and the other carries predominantly live load. The uncertainty associated with dead loads and live loads is not the same. This uncertainty is accounted for when using LSD methods, but not with WSD methods.

Load and resistance factors, or partial safety factors (PSF) can be determined using probability and reliability methods; calibration procedures, which are used to achieve results similar to WSD; and also from engineering judgement. The PSFs used in design codes are specified for each limit state and are based on target values of reliability or acceptable probability of failure. Design codes will not cover all design situations.

Engineers can apply the LRFD method to other design problems, although it is not recommended to use PSFs outside the design codes for which they were derived. The use of PSFs from other codes is not recommended because the assumptions made during code writing may not be applicable to the project at hand. For non-codified engineering projects, the engineer will have to determine unique PSFs using probability and reliability methods.

<u>Partial safety factor method</u>

The partial safety factor (PSF) method is an extension of the LRFD and much of the above discussion on LRFD is directly transferable to this method. The PSF method utilizes multiple load and resistance factors applied to different components of the design calculation as implied in Equation 2-5. This method is used in Australia (Day, 1997) and is not unlike the factored strength method used in Europe. In general, the partial factors increase the load effects and reduce the resistance of the structure. In geotechnical design, different factors may be applied to the separate variables including shearing resistance, cohesion, density and loads. Chapter 5 describes a new application where PSFs were used in an unusual geotechnical project for which no previous practical or design experience existed. This application forms the principal original contribution of this thesis project.

The general equation is:

[2-5] $$\Sigma\phi_i R_{ni} \geq \Sigma\alpha_i S_{ni}$$

where

$\Sigma\phi_i R_{ni}$	- summation of the factored resistances;
ϕ_i	- resistance factor corresponding to a particular resistance, $R_{ni;}$
R_{ni}	- nominal resistance determined from engineering analysis;
$\Sigma\alpha_i S_{ni}$	- summation of the factored overall load effects;
α_I	- load factor corresponding to a particular load, $S_{ni;}$
S_{ni}	- specified load component of the overall load effects;
i	- represents various types of resistances (shear resistance, density, bearing capacity) and loads (dead load, live load, wind load).

LSD using worst credible values

Limit state design using worst credible values was proposed by Simpson et al. (1981) to overcome the apparent shortcomings of various design methods including working stress design, partial factor methods, probabilistic methods and design based on worst attainable values. The authors believed that none of these methods could be used consistently in geotechnical or structural design. Two objections to the use of partial factors in geotechnical design include:

- The degree of certainty with which a given geotechnical parameter, such as undrained shear strength can be assessed varies significantly from one project to another. The use of prescribed partial factors applied to mean or characteristic values of the parameters are therefore not adequate for deriving design values intended to accommodate the variability.
- It is not sensible to apply factors to water pressures and geological uncertainties.

Instead of using prescribed factors, Simpson et al. (1981) suggested that the designer think about the worst circumstances that might arise and express these in the design parameters. They distinguish the difference between expected parameter values and worst credible parameter values. Expected values are those occurring in the average situation which will govern the real behaviour and may either be constant with time (shear strength, for example) or vary with time (groundwater levels, for example). Worst credible values are values of loads and material properties that are the worst that the designer could realistically believe might occur. The worst credible value is not the worst that is physically possible, but rather a value that is very unlikely to be exceeded. This is basically the same objective that can be achieved by applying PSFs to characteristic or nominal values of strength parameters and other uncertainties. No explicit level of safety (reliability), such as probability of failure, was defined for this method.

Without going into great detail, the basis of the method is to define a limit states value for each variable and to demonstrate that the limit state will only just occur with these values. One of two constants are applied to each variable, dependent on how the uncertainty of each variable affects the design. The method considers uncertainty in parameters, but not model uncertainty. Given that the designer must select the worst credible value for each variable there is significant room for variability between designs by different engineers.

The method does not appear to provide any greater consistency than other LSD methods because, as in any method used for geotechnical design, the potential for design variability between different engineers is significant. One of the main reasons for these differences relates to how the geotechnical data are interpreted. The method suggests there are ranges of values that may have upper and lower bounds. Identifying these limits is an important aspect of any design. There is no benefit in using factors of any kind that yield conditions that are very rare or even physically impossible, such as a saturated unit weight of normally consolidated clay equal to 25 kN/m^3.

<u>LSD using extreme values</u>

Bolton (1981) proposed a limit state design method based on extreme values. He suggested that design approaches using factors of safety and probabilistic design methods should not be used to demonstrate safety of geotechnical structures. The general method proposed involves checking designs for occurrences of limit states when all parameters are assigned their worst obtainable values and using conservative models. Under these conditions, Bolton feels that no factors of safety are necessary. Simpson et al. (1981) suggested that the proposed method is pessimistic and would likely result in overly expensive structures. In addition, the probability that all the parameters would be operating at the extreme values simultaneously is expected to be very low. Becker (1996a) suggests the method has some merit because engineers would be required to think about worst case scenarios that might arise. Use of extreme values may be useful in certain situations where some conditions or engineering properties are not well known. Again, this shows the importance of, and need for, engineering judgement and experience. The use of extreme values may be appropriate when other LSD methods are used outside of design codes. An example of this approach might involve assuming an extreme value for a variable that is difficult and costly to measure and has a relatively small impact on the design.

<u>Reliability-based design</u>

Becker (1996a) discusses reliability-based designs (RBD) and how they relate to methods such as LRFD. In RBD, the parameters are treated as random variables instead of constant deterministic values. The measure of safety is the probability of failure that can be computed directly if the actual probability density functions or frequency distribution curves are known, or measured for the loads and resistances. The probability of failure is related to the shaded area representing the overlap between the load and resistance curves shown in Figure 2-2.

As for all design methods, RBD cannot account for gross errors including misinterpretation of site conditions, incorrect assessment of geotechnical properties or the use of inappropriate models by the geotechnical engineer (Phoon et al. 1993). RBD has some significant advantages such as being more realistic, rational, consistent and widely applicable. However, the biggest disadvantage is that generally large amounts of data are required to implement RBD procedures successfully and the computational procedures are more complicated than those in deterministic design methods. Few engineers are adept at using probabilistic and reliability methods. Thanks to modern computers and software, RBD procedures are becoming easier to implement, as will be shown in Chapter 5.

There are three levels of RBD or probabilistic design (Becker, 1996a). Level 3 is a fully probabilistic method that requires the actual probability distribution curves be known or measured for each random variable for loads and resistances. The biggest disadvantage of the Level 3 method is the amount of time required to complete the analysis and typically the engineer will not have all the required information to complete the analysis. This method may be useful for large projects where the consequences of failure are high. It may achieve more prominence in the future as the necessary databases are assembled, particularly in locations where localized geologic conditions are fairly uniform. Examples might include probabilistic analysis of stability of riverbanks in Winnipeg, highway cuts in clay-till, and spoil heaps in open-pit mines. A Monte Carlo simulation could be used to complete a Level 3 reliability design.

The Level 2 probabilistic method is known as an approximate probabilistic method. It does not require that the actual probability distribution curves be known, but the shape of the curves is required. Statistical parameters from collected data can be used to describe or approximate the distribution of the variables. It is common to assume that the variables are statistically independent variables and have either a normal or log-normal distribution. A special case of the Level 2 method is the second moment probabilistic method. The method considers the random nature of the variables and is based on the two moments of the mean and the coefficient of variation of the loads and resistances. Safety is defined by the reliability index, β (MacGregor 1975). While there are several ways of setting up the problem, one of the simplest is to determine the mean and standard deviation of the "safety margin". The safety margin is equal to the resistance minus the load effects at the ULS. The reliability index is defined as the number of standard deviations that the mean of the safety margin distribution lies above zero or the failure limit (Allen 1975). The reliability index has been related to probability of failure as shown in Table 2-1. Becker (1996b) provides a good discussion on the reliability index. Equations 2-6 to 2-9 (Ravindras and Galambos, 1978 and Alen and Jendeby, 1993) are used to calculate load and resistance factors using the second moment method. MacGregor 1975 and Becker 1996b describe the assumptions and theory behind the calculation of load and resistance factors. A Monte Carlo simulation could be used to complete a Level 2 reliability design.

[2-6] $\lambda = \delta\, e^{\beta\theta V}$ (log-normal distributions)

[2-7] $\phi = \delta\, e^{-\beta\theta V}$ (log-normal distributions)

[2-8] $\lambda = \delta\,(1+\beta\theta V)$ (normal distributions)

[2-9] $\phi = \delta\,(1-\beta\theta V)$ (normal distributions)

where:

λ - load factor
ϕ - resistance factor
δ - bias factor = mean value / nominal value
β - reliability index
θ - separation coefficient (typical range of 0.6 to 0.8)
V - coefficient of variation = standard deviation / mean

The Level 1 or semi-probabilistic method uses separate load and resistance factors or partial safety factors determined from a Level 2 reliability analysis. The LRFD is an example of a design method based on Level 1.

As shown above, there are a variety of methods that can be used with LSD concepts to account for uncertainties in design. Compared with WSD, LSD is a more rational approach that can provide more consistent designs and associated levels of safety. In non-codified classes of engineering problems, we may need a number of 'tools' to complete a LSD. These tools include engineering judgement, knowledge of probability and reliability methods and knowledge of realistic extreme values (upper and lower bounds) applicable to the parameters in the design.

The buoyancy analysis of the Shoal Lake Aqueduct presented in Chapter 5 of this thesis shows how the partial safety factor and reliability-based design methods can be used as a platform for a non-codified design. The work required some assumptions to be made because it was not possible to determine partial safety factors for all the parameters. The work also shows why it is not wise to use partial safety factors from design codes for anything but their intended applications. This is particularly true for existing infrastructure from which variables can be measured directly.

2.4.5 General Concerns Regarding The Use Of Limit States Based Design Codes In Geotechnical Engineering

All evidence that has been found in this review supports the use of LSD concepts in geotechnical engineering. However, some authors, for example; Boden 1981; Bolton 1981, 1993; Semple 1981, Simpson et al. 1981, Fleming 1989 and Day 1997, have strong opinions about how uncertainties should or should not be included in the LSD process. The majority of the concerns appear to be with the use of prescribed PSFs. Many of the authors felt that the inherent variability of soil conditions from site to site cannot be properly accounted for using prescribed PSFs and that there is not enough information available to determine appropriate values for PSFs. Fleming (1989) felt that prescribing universal factors without nominating a particular method would be inappropriate and that prescribing design methods would take away the engineers use of knowledge and experience.

Boden (1981) pointed out that prescribed PSFs applied indiscriminately to soil properties might even define soils that could not possibly exist in nature. Boden also felt that any changes made to existing design methods must retain a framework within which past experience and engineering judgement can be readily applied, particularly where empirical design is used. An example of this is footing design using Standard Penetration Testing results. Simpson et al. presented two objections to the use of prescribed partial factors. The first being that the degree of certainty with which a given geotechnical parameter, such as undrained shear strength, can be assessed varies significantly from one project to another. Prescribed PSFs applied to mean or 'characteristic' values of the parameters are therefore not adequate for deriving design values intended to accommodate the variability. Second, it is not sensible to apply PSFs to water pressures and geological uncertainties.

Semple (1981) presented four criticisms of PSFs. One, due to the number of PSFs, the method is cumbersome to use. Two, results of the analysis must fit with existing experience and therefore do not produce substantial differences in overall safety factors. Three, splitting the global factor of safety (GFS) into components associated with loads and resistances introduces the possibility that one or more functions of the original GFS may have been omitted. Finally, when failures occur, it is almost always due to serious errors or unforeseen conditions. It seems that this last statement would apply to all design methods. Semple felt that failures due to excessive variation of recognized parameters are rare and should not be given undue emphasis by focusing attention on PSFs. He also suggested that statistics can become a substitute for trying to understand inherent, non-random variations in soil characteristics. In contrast, Semple argued that statistics should help enhance our understanding of geologic uncertainty and variability, a process that is more rational than drawing conclusions from raw field data.

More recently, Day (1997 and 1998) echoed these concerns. Day felt that it is impossible to specify material factors or capacity reduction factors based on probability considerations that can be applied in general to all sites and at the same time take into consideration the variability of all parameters. He was concerned that the calibration process, particularly for foundations, may be wrongly conceived. Data used to determine values of PSFs for resistance are not appropriate because while LSD separates strength and deformation, WSD does not do so explicitly. Yet the WSD results are used to determine PSFs that can be used in LSD. It is noted again that WSD for foundations applies a high global factor of safety (2.5 – 3.0) to the strength (bearing capacity) equation to limit settlements.

The references quoted in previous paragraphs all show that PSFs are not solidly based on probabilistic models or reliability calculations, as originally proposed. Becker (1996b) showed how PSFs for a LSD code for foundations (NBCC 1995) were calibrated by fitting with WSD and using reliability methods. Day concluded that many of the new codes are just a different method for achieving the same result as previous "good practice". In his opinion, these new codes tend to distract the engineer with numerical modeling rather than allowing the engineer to focus on the real behaviour of the system.

More recently, Green and Becker (2000) identified some specific problems and issues with LSD in geotechnical engineering that are much different than the concerns presented in earlier papers. They suggested that change is generally not welcomed by many, especially when it involves switching from a well-known system (WSD) that works to lesser-known system (LSD) that is not well understood by many engineers. The lack of acceptance of LSD is likely due to inadequate education on LSD and its benefits. While much effort has been spent on the mysteries of probability theory, Green and

Becker suggested that engineers should promote a fundamental understanding of how to identify and assess the limit states associated with different classes of practical problems. This would then be followed by consolidating the analysis used for designs based on uncertainty and risk. Geotechnical engineers need to enhance site investigation methods and the post-processing of field data. They also need to understand and embrace the fundamental principles of LSD and learn that LSD is much more than the hidden application of probability and reliability theory. Green and Becker recognized that more guidance is required in determining appropriate geotechnical design values to be used in LSD. However, the selection of design values is also a function of the site characterization program and geotechnical engineering experience and judgement. Particular classes of problem that need further development include predicting settlements and soil-structure interaction.

None of the references used for this review disputed the need for using LSD. Many, however questioned how uncertainty should be accounted for. The obvious problem with all design methods, including both LSD and WSD methods, is how to select design parameters and account for geotechnical or geologic uncertainty. In many classes of problems, geotechnical engineers have sufficient experience and judgement using WSD to be able to proceed with an acceptable level of comfort. Use of LSD methods directly addresses some of the shortcomings of WSD such as uncertainty in loading. Other uncertainties remain, principally those associated with geologic uncertainty and will never be easily solved. Engineering experience will play a significant role in bridging the gap between theory and the real world. Engineers will adapt to new methods once they start using them and gain experience. There is much more room for growth and improvement with LSD methods, including the PSF and LRFD methods, than there is with WSD. Engineers need to be looking for and adopting new and better methods that enhance and rationalize previous experience and judgement.

A smooth transition from WSD to LSD is required to promote the use of these methods in geotechnical engineering. Engineers know that existing methods work well in most cases, but we still don't really know what the level of safety is or how the global factor of safety is distributed amongst the different variables. Geotechnical engineers have been using less than satisfactory methods for years, especially for soil-structure interaction problems, because they are comfortable with them. For new problems and innovative technologies, we need to improve the methods of analysis. We also need to be able to provide recommendations to our structural engineering counterparts that can be used directly in structural design. The author suggests that the catalyst for assisting in achieving these objectives may be LSD.

2.5 APPLICATION OF LIMIT STATES DESIGN METHODS TO NON-CODIFIED ENGINEERING PROBLEMS

In many design situations in geotechnical engineering, no design code exists, particularly for existing infrastructure remediation projects, for example, the buoyancy assessment project for the Shoal Lake Aqueduct that forms the principal case study in this thesis document. In such situations, LSD methods based on partial safety factors or reliability methods can be used to account for uncertainties. Concerns regarding the use of prescribed partial safety factors that are not based on probabilistic methods were identified in Section 2.4.5. In some design situations these concerns can be overcome by collecting data and using reliability methods (Levels 1, 2 or 3) to provide the desired level of safety. This is especially true when engineering properties can be directly measured for existing infrastructure. This was the case for the Shoal Lake Aqueduct. In projects like this there are potential cost savings for remedial work completed on existing infrastructure because minimal allowance needs to be made for construction tolerances. This may permit the use of higher partial resistance factors and lower partial load factors with corresponding economics.

No general guidelines are available on how to complete a limit states-based design using reliability methods when no design codes have been developed. Each project will be unique, but application of the approaches outlined in this thesis shows how valuable this design tool can be to practicing engineers.

Application of LSD methods to non-codified engineering problems requires engineers to identify all possible limit states and associated variables and as well, an acceptable level of safety. Appropriate engineering models are required to assess each limit state. Prior to undertaking a field investigation to collect required information on the variables, a simple Monte Carlo simulation model can be set up to determine which variables appear have the greatest influence on the various limit states.

Computer programs such as @-Risk (Palisade Corporation, 2002) can be used to complete a Monte Carlo simulation. This work requires that a reasonable statistical distribution for each variable be established. This is done on the basis of available information, engineering judgement and experience. Statistical output from the model permits the engineers to evaluate which parameters are most influential. A field program can subsequently be developed to ensure that sufficient data are collected for each variable. Post-processing of the field information is an important phase of the work, requiring statistical methods, engineering judgement and experience. Post-processing of field and lab data may include deleting some results from the data set due to sample disturbance or the grouping of data from large sites. Neither of these involve statistics but do require engineering judgement. Numerous papers have been written on statistical methods to help with data interpretation, for example Phoon and Kulhawy, 1999 and the papers from the ASCE Conference "Uncertainty in The Geologic Environment: From Theory to Practice."

In this class of problem, there is a good chance that one or more of the variables may not be measurable. Under these situations, some of the ideas from the preceding discussion on LSD methods, for example, worst credible or extreme values, can be used with engineering judgment to determine appropriate design values that may not require the use of partial safety factors. Examples might be groundwater elevation or unit weights of backfill. In both cases there is likely a minimum and/or maximum value that can be reasonably assumed on the basis of engineering judgement, experience and site knowledge.

For example, during the buoyancy assessment project for the Shoal Lake Aqueduct it was not feasible to determine the variability in ground water levels. On the basis of observed site conditions and measured groundwater levels, it was assumed that groundwater levels in most areas along the aqueduct were high enough to assume complete submergence of the aqueduct and the backfill. This may seem like a logical assumption. However, considering the cost associated with mitigating buoyancy problems, careful thought was required in making this 'logical' assumption. There were at least two stretches of the aqueduct where the assumption of complete submergence of the pipe and overlying backfill was not applicable. In these locations, hydrologic modelling was used to determine that water levels would not likely rise above the top of the pipe. This would leave a significant portion of the backfill non-submerged, resulting in fewer remedial repairs and substantial savings in construction cost.

Chapters 3 and 4 of this thesis report respectively on the initial design and construction of the Shoal Lake Aqueduct in the years 1914 to 1919; and on a buoyancy assessment program undertaken from 1994 to 1999 by UMA Engineering Ltd. in partnership with CH2M_Gore and Storrie Ltd. and the City of Winnipeg. Chapter 5 shows how three LSD methods were applied to reduce the risk of buoyancy of the Shoal Lake Aqueduct.

3.0 SHOAL LAKE AQUEDUCT

3.1 BACKGROUND

The Shoal Lake Aqueduct (SLA) is a 157 kilometre (97.5 mile) long concrete pipe that transports up to 385 million litres per day (MLD) of potable water from Indian Bay to Winnipeg, Manitoba, Canada. Winnipeg is located about 100 kilometres north of the Canada-United States border at the junction of the Assiniboine River and the Red River. Indian Bay is located about 150 kilometres east of Winnipeg on the west side of Shoal Lake (Figure 3-1). The aqueduct was constructed between 1915 and 1919 using cast-in-place concrete and pre-cast concrete pipe sections.

Much of the following information about the design and construction of the SLA comes from original manuscripts, reports, drawings and archival photographs. The principal sources of information include the following references: GWWD 1918, GWWD 1914, Feurtes 1920 and Chase 1920a and 1920b. GWWD is the Greater Winnipeg Water District. These archives are located in the Shoal Lake Aqueduct Resource Center at the City of Winnipeg's Water and Waste Department.

During the 40 year period of time prior to construction of the aqueduct, Winnipeg's rising population resulted in increasing demands for a bountiful water supply of a quality suitable for human consumption and industrial uses. In 1874 Winnipeg's population was 1,869. By 1890 the population had increased to 23,000; by 1910 it was 132,720 and by 1913 the population had reached 215,000. Before construction of the aqueduct, water for human consumption and industrial purposes was obtained from either groundwater wells or the Assiniboine River. In the years before 1882 the water supply for Winnipeg consisted of a number of groundwater wells. The water was distributed to the consumers by horse drawn carts or sleighs in winter. Between 1882 and 1900 raw water was obtained from the Assiniboine River but was filtered before entering the water mains. From 1900 until 1919, when the aqueduct was first commissioned, water was taken from an artesian well system located within the city limits, in the general area of McPhillips Street and Logan Avenue (GWWD, 1907). Although water provided by the artesian well system was palatable, this water was very hard and resulted in high maintenance costs for industrial users. In addition, local groundwater levels were depleted as the demand for water increased.

The need for a permanent water supply was recognised as early as 1883 but it was not until 1905 that further action was taken. Two studies were undertaken to determine suitable sources of soft water, the first recommended the Winnipeg River and the second recommended Shoal Lake. By 1913 Shoal Lake was selected as the preferred source of a permanent water supply since it was of high quality and quantity and could be transported by gravity in quantities much larger than those considered to be available from the Winnipeg River source (Feurtes, 1920).

Economics of construction played a large roll in deciding where to draw the water out of Shoal Lake. The Intake was located on Indian Bay to provide the most economical route even though difficult construction conditions were expected over the first 19 kilometres of aqueduct alignment directly west (downstream) of the Intake. In general, the water in Indian Bay is clear but near the proposed Intake location the water was highly coloured. Turbid water from Falcon River discharged into Indian Bay resulting in highly coloured water near the proposed intake location. This problem was alleviated by diverting Falcon River into Snowshoe Bay. The diversion was accomplished by constructing a dyke across the west side of Indian Bay and a canal across the promontory separating Indian Bay and Snowshoe Bay (Figure 3-2). These works are known as the Falcon River Diversion. The colour of water in Indian Bay improved almost immediately after the Falcon River Diversion was completed.

Approximately five and one-half (5 ½) years after selecting Shoal Lake as the preferred water source, the aqueduct was commissioned and the first waters were discharged into the reservoirs located on McPhillips Avenue. The major events that took place during this period of time include:

- 6-Sept.-1913) the Greater Winnipeg Water District (GWWD) adopted the report recommending Shoal Lake as the preferred water source.
- 8-Sept.-1913) the City Council of Winnipeg also adopted the report recommending Shoal Lake as the preferred water source.

- 1-Oct.-1913) 97 percent of the legally qualified ratepayers of the City voted to accept the by-law creating a debt of $13.5 million dollars required to construct the aqueduct.
- 2-Oct.-1913) an order in Council of the Province of Ontario was passed permitting the GWWD to draw from Shoal Lake, up to 454 million litres of water per day.
- 20-Oct.-1913) five field parties were sent out to gather preliminary survey data.
- 15-Jan-1914) the International Joint Commission approved the application permitting the use of water for the GWWD from Shoal Lake up to 454 million litres per day.
- February, 1914) aqueduct route selection completed.
- March, 1914) construction began with clearing of the right-of-way, installation of a telephone line, railway line construction and construction of the Falcon River Diversion.
- October, 1914) construction contracts awarded for 137 kilometres (85 miles) of aqueduct east of Mile 12.5. (Notes: Mileage refers to the distance from McPhillips Street Reservoir (Mile 0) to the Intake (Mile 97.5). Separate contracts were later awarded for aqueduct construction west of Mile 12.5.
- May, 1915) aqueduct construction from Mile 12.5 to Mile 97.5 was started.
- End of 1918) aqueduct construction complete (as required by the terms of the Contract).
- 26-March-1919) the first water flowed into the McPhillips Street Reservoir.

The reference system used to locate and identify specific aqueduct features is 'aqueduct mileage'. The aqueduct mileage increases in the upstream direction from the McPhillips Street Reservoir located at Mile 0 of the aqueduct and terminates at Mile 97.5 (the Intake) on Indian Bay (Figure 3-3). This reference system will be used throughout this thesis to identify the location of specific aqueduct features. The main components and the respective mileage's are listed below.

- McPhillips Street Reservoir – Mile 0
- Branch I Aqueduct (BR1)– Mile 0 to Mile 12.5
- Deacon Reservoir – Mile 12.5
- Shoal Lake Aqueduct (SLA) – Mile 12.5 to Mile 97.5
- Intake – Mile 97.5

From Mile 0 to Mile 12.5 the aqueduct was constructed using pre-cast pipe placed in a shored trench (Figure 3-4). The construction technique used was generally straightforward and accomplished without major difficulty. The 'cut and cover' construction scheme (Figure 3-5) used between Mile 12.5 and the Intake at Mile 97.5 was simple in design but many challenges were encountered during construction. The aqueduct construction from Mile 12.5 to the Intake will be discussed in detail as it pertains directly to the area for which the buoyancy assessment was undertaken and that forms the principal technical component of this thesis document. Some of the problems were anticipated, based on soil investigations, while others were not discovered until the first year of construction. In order to alleviate some of the foreseen construction problems, the Greater Winnipeg Water District assumed multiple roles during construction including that of Contract Administrator and Sub-Contractor.

3.2 AQUEDUCT CONSTRUCTION

3.2.1 Aqueduct Features

The aqueduct and all the appurtenant structures were constructed between May 1915 and December 1918. The general features of the aqueduct consist of about 137 kilometres (85 miles) of gravity flow pipe, 20 kilometres (12.5 miles) of pressure pipe, six inverted siphons at river crossings, five overflow structures, a tunnel under the Red River and a series of drainage siphons. Although the aqueduct was designed to be operated with a reservoir at Mile 12.5 this component was not constructed until 1972. This project represented a major undertaking for the small Manitoba community in the early 1900's.

The Branch 1 Aqueduct (BR1) (Mile 0 to Mile 12.5) was constructed with precast, reinforced concrete lock-joint pipe (Figure 3-6). From the McPhillips Reservoir (Mile 0) to the west side of the Red River (Mile 3.5) the pipe segments are 3.05 metres long and have an inside diameter of 1.22 metres. From the eastside of the Red River (Mile 3.5) to Deacon Reservoir the pipe segments are 2.43 metres long and have an inside diameter of 1.68 metres.

The BR1 on the west and east sides of the Red River is connected by the Red River Tunnel. The Red River Tunnel consists of a 1.52 metre diameter cast iron pipe located in bedrock about 6.1 metres below the river bed. Vertical shafts on the west and east sides of the river complete the tunnel. The annulus between the bedrock and the cast iron pipe was filled with concrete.

The SLA (Mile 12.5 to the Intake at Mile 97.5) is a two piece, cast-in-place concrete structure through which water flows by gravity to Deacon Reservoir. Before the Deacon Reservoir was completed water flowed directly to the McPhillips Reservoir by gravity. This section of aqueduct is essentially a covered concrete lined ditch consisting of an upper arch resting on a flattened inverted arch base referred to as the invert (Figure 3-7). Reinforcing steel was used sparingly due to the high cost and reduced availability during wartime (World War 1). Only 91 metres of elevation change (grade) was available between Mile 17 and Mile 97.5. Eleven different aqueduct section sizes were required, that is the internal dimensions of the SLA were varied to suit the natural slope (grade) of land along the aqueduct alignment. The smallest aqueduct section measured 1.950 metres wide by 1.632 metres tall (grade = 0.1537%). The largest aqueduct section measured 3.277 metres wide by 2.743 metres tall (grade = 0.011%).

At six locations along the route of the aqueduct it was necessary to cross rivers using inverted siphons. These rivers include: the Seine River (Mile 4), the Brokenhead River (Mile 41), the Whitemouth River (Mile 64), Birch River (Mile 74), Boggy River (Mile 78 and Mile 82) and the Falcon River (Mile 97). At these locations the aqueduct was constructed of reinforced concrete pipes approximately 2.43 metres in diameter.

Overflow weirs were constructed on the upstream end of the inverted siphons located at Miles 42, 64, 74, 83 and also at Mile 17. The weirs are rectangular windows cut into one side of the aqueduct arch just above the waterline at maximum flow. The weirs were designed to discharge the maximum aqueduct flow of 385 MLD. The weirs protect the non-reinforced aqueduct arches from surcharging caused by a partial or complete blockage in the aqueduct, or by careless operation of the intake gates at Mile 97.5.

In general, the aqueduct backfill berm is located above prairie level. To allow for natural drainage across the line of the aqueduct about forty inverted drainage siphons were built below the aqueduct invert (Figure 3-8). These siphons have a square cross-section and range in size from single sections measuring 0.91 metres by 1.22 metres sections to triple barrelled sections with each barrel measuring 1.83 metres by 2.06 metres. The siphons consist of an inlet and outlet inclined at approximately 45 degrees connected by a horizontal segment below the aqueduct.

3.2.2 Shoal Lake Aqueduct Design

The SLA was constructed using eleven different aqueduct section types. These sections are identified by a two letter code. For example, BO. The letter B refers to the internal size of the aqueduct, which varied to suit the available grade along the route of the aqueduct. The second letter refers to the type of invert constructed to suit the foundation conditions along the route of the aqueduct. There are a couple of situations where a third letter was used to indicate a change to the arch, BPM for example. The third letter, M, indicates that a modified arch was constructed. Only the first letter, which describes the aqueduct section size, will be used when the discussion is irrelevant to the type of invert constructed. For example, the S section was constructed from Mile 89.09 to the Intake at Mile 97.5. If the invert type is relevant to the discussion then both letters will be used. The various aqueduct sections constructed between Mile 85.0 and the Intake at Mile 97.5 are summarized in Table 3-1. These sections are presented because buoyancy assessment of the aqueduct was conducted in this area.

The arch design of the aqueduct was influenced by many factors including foundation rigidity, economy, internal and external water pressures, backfill weight and pressure, frost protection and practical construction methods. The stresses within the arch were determined using a graphical thrust line procedure (Figure 3-9). Prevention of cracking in arched structures requires a virtually unyielding foundation, invert deflections as little as 0.6 millimetres can result in cracking of the arch crown and spreading of the arch legs. Increasing the thickness of the arch up to three times that required for a perfectly rigid foundation would not prevent cracking under uneven settlement of the footings of the arch (Feurtes 1920). Therefore, the engineers assumed that a rigid foundation could be provided during construction. They were well aware of the challenging foundation conditions but realized that economy in design was important for a young community like Winnipeg. To design the aqueduct on the principle of 'no risk' would have rendered the project financially impossible. The engineers realized that repairing cracks due to settlement was much more cost effective than providing perfectly rigid foundations or thickened arches. Thickening of the arches and inverts by 25 millimetres would have required an additional 27,000 m^3 of concrete. The additional concrete would have increased construction costs by approximately $500,000, a very considerable sum in the early 1900's.

The inverts for the aqueduct had to be redesigned following the first year of construction (1915) due to longitudinal cracking observed in about 20% of the inverts (approximately 4 kilometres) that had been constructed and backfilled. During the first year of construction (1915), a non-reinforced invert, 150 mm thick was used exclusively (Figure 3-10). This invert is also known as the standard invert or an 'O' type invert. The longitudinal cracking was attributed to two factors. The first factor was poor or complete lack of foundation soil compaction along the outer edges of the invert, that is, directly below the arch legs. This resulted in small settlements and hairline cracks once the full backfill load was applied. Settlements as small as 1 millimetre could result in hairline cracks along the invert centerline. Second, inverts constructed on soft/loose (compressible) foundation soils resulted in relatively large settlements and cracks. The problem of hairline cracking was largely corrected by requiring the Contractor to place a thin bed of gravel along the outer edges of the invert foundation and compacting/ramming this material into the foundation soil. Settlements due to compressible foundation soils were minimized by reducing the bearing pressure across the invert. This was achieved by using either a wider invert or a thickened reinforced invert. The thickened reinforced invert prevented cracking and therefore minimized settlement at the outside edges of the invert. These invert type are discussed below.

A testing program on full-scale inverts was undertaken in 1916 to investigate the problem of invert cracking. This represents a very early use of full-scale field tests to confirm design assumptions. Until the results were available, two additional invert designs were used exclusively in order to reduce the potential for cracking. On all solid foundations, whether compressible or not, the invert used in 1915 (Figure 3-10) was to be used but with a slight modification. The outside edges of the invert were extended 200 millimetres past the outside edge of the arch leg to increase the bearing area (Figure 3-11). On all questionable foundation soils a thickened, reinforced invert was to be used (Figure 3-12). The invert was thickened by 50 to 215 millimetres depending on the aqueduct section size. These two invert types were used throughout the 1916 construction season.

The invert testing program was undertaken near Mile 13 of the aqueduct during the 1916 construction season to aid the Engineer's understanding of actual bearing pressure distribution on the invert (Figure 3-13). The testing was conducted on a total of 16 invert sections representing nine different invert cross-sections. Each test invert was 600 millimetres wide and was loaded using steel rail sections placed upon 600 millimetre high pedestals shaped like the lower extremities of the arch. Settlement profiles across the invert were recorded as the loading increased and also at the first sign of cracking. Of the nine different section types tested, one was the same as the standard invert section used in 1915 (Figure 3-10) (built as a control section), seven were reinforced but had different concrete thickness and different steel spacing and diameter. The remaining invert test section was similar to the standard invert except that it was constructed 300 millimetres wider on each side.

The standard invert section (Figure 3-10) that was built as a control section performed in a similar manner to those constructed in 1915, indicating that the test method used was a good approximation of the field conditions. All but the thickest (285 millimetres) reinforced invert sections had 2 to 6 cracks develop at loads approximately equal to the arch weight + backfill weight + water pressures. Similar loads on the 150 millimetre thick, widened invert section only resulted in a single crack. Cracks in the non-reinforced invert sections could be repaired in a manner that made them as strong as the originals while the reinforced sections were much more difficult to repair due to the presence of multiple cracks and reinforcing steel. Cracking of the reinforced sections would also result in the risk of corrosion of the steel reinforcement. Based on the results of the invert test, three invert sections were designed for use during the remaining two years (1917 and 1918) of the aqueduct construction program. These invert sections are shown on Figure 3-14 and are discussed below.

Figure 3-14 shows the three invert sections that were selected to minimize construction costs. Using a single invert for all foundation conditions would have resulted in a much higher construction cost due to the additional materials required. Having a selection of three inverts provided the field Engineer's with some flexibility to choose an invert section that was appropriate for varying foundation conditions as they were encountered. The original invert section used in 1915 (Figure 3-10 and Figure 3-14, invert type 'O') was to be used on what was considered '*solid or the best foundations*'. A thick, reinforced invert section (Figure 3-14, invert type 'A') was to be used on what was considered to be '*the worst*' foundation conditions and for all other foundation conditions classified as '*better than the worst but poorer than the best*' a widened invert section was to be used (Figure 3-14, invert type 'E'). The inverts shown in Figure 3-14 are for the 'B' aqueduct section size as denoted by the first letter in the two letter identification code. These invert section types were designed for each of the aqueduct section sizes that were constructed in 1917 and 1918. Table 3-1 summarizes the dimensions of the inverts constructed between Mile 85.0 and the Intake at Mile 97.5.

One additional invert section was designed, not to prevent cracking, but to prevent flotation or buoyancy of the aqueduct. This 'gravity' invert, was used only under the following conditions:

- groundwater levels above the aqueduct crown
- light weight backfill (for example, organics or peat)
- porous foundation soils

The gravity invert, shown in Figure 3-15, was up to 790 millimetres thick, depending on the aqueduct section size. The invert thickness was reduced when an '*impervious*' foundation, such as clay, was encountered before excavation below grade reached the maximum invert thickness. This invert design was determined to be the most feasible and economical means of preventing flotation of the aqueduct under the above conditions.

The engineers were well aware of sulphate attack on concrete structures built on Winnipeg (Lake Aggasiz) clays. The area of major concern for the aqueduct lies within the first 42 kilometres of the aqueduct alignment (Mile 0 to Mile 26) where both the foundation and backfill soils are Winnipeg (Lake Agassiz) clay. Sulphate-resistant concrete was not available at the time the aqueduct was constructed but the engineers were aware of the conditions that would lead to sulphate attack. The engineers initially assumed that hard, dense concrete with a smooth exterior surface would help to resist disintegration due to sulphate attack. This assumption proved incorrect and in 1917 a system of vitrified clay tile underdrains was incorporated into the aqueduct construction plans, where required. This was done to keep the concentration of dissolved sulphates in the groundwater to a minimum. A considerable portion of the aqueduct constructed west of Mile 26, prior to 1917, was uncovered and underdrains placed as close as possible to the outside edge of the invert.

3.2.3 General Construction Scheme

The cut and cover construction scheme shown in Figure 3-5 was straightforward in principle and generally all operations were kept within an 800 metre long work area. Excavation by machine was allowed to within 150 millimetres of the final invert foundation grade. The final 150 millimetres was trimmed manually using templates as a guide to ensure proper geometry of the invert. Final trimming of the foundation soil was not allowed until the final hour before the concrete was to be placed. This was done to prevent drying and cracking of the foundation soil. The first inverts were cast in lengths of 4.57 metres but spaced every 9.14 metres (Figure 3-16). The 4.57 metre gaps left open between the first inverts were filled in after sufficient curing time had passed, thus minimizing shrinkage effects. The first inverts had copper waterstops embedded into the concrete at both ends of the invert. A wooden waterstop was embedded into the shoulder of the inverts, the area upon which the arch legs were to rest. To ensure a good bond between the arch and invert, the shoulder of the invert was prepared using wire brushes leaving a clean, roughened surface.

Figure 3-17 shows that the arches were also cast in an alternating pattern but were typically cast in 13.7 metre lengths. They were cast using a system of inner and outer steel forms, which were moved from site to site using a temporary tramway system. The first arches had copper water stops embedded into the concrete at both ends of the arch.

Where possible the lower 1.2 metres of backfill between the arch leg and the trench wall was placed in thin lifts and carefully compacted to provide additional lateral stability to the arch (Figure 3-18). This has been referred to in Chapter 5 as the 'compacted backfill'. The remainder of backfill was placed by machine to a depth of 1.2 metres over the aqueduct crown when mineral backfill soils were available or to a depth of 1.5 metres when only organic soils were available for backfill.

3.2.4 Role of the GWWD and the Contractor

The GWWD desired to have the aqueduct constructed and operating as quickly as possible. To accomplish this they adopted a larger role than is currently common. Early in the planning stages, it became apparent that the work should be tendered in five moderately sized contracts, each about 27 kilometres (17 miles) in length, rather than one large contract (GWWD 1914). This allowed more Contractor's to bid the job. Due to poor site conditions (particularly east of Mile 40) and a lack of good quality sand and gravel sources within each Contract, the GWWD realized that they should also provide transportation and aggregate to the Contractor. The GWWD also decided to supply cement to the Contractor at cost. The contracts were structured in such a way that the GWWD was responsible for transportation of all supplies and personnel to the construction sites and for the provision of approved cement and concrete aggregate. In effect, the GWWD became a Sub-Contractor to the principal Contractor. The Contractor's work included:

- all necessary excavation
- foundation preparation
- construction of the aqueduct
- backfilling of the aqueduct
- supply of all materials except cement, sand and gravel, venturi meters, miscellaneous brass and bronze pieces and transportation of the above.

The GWWD's role was to provide approved cement and concrete aggregate, in addition to other supplies, to the Contractor on a timely basis. The GWWD completed numerous tests to achieve an economical concrete mix that also met the criteria of low permeability, high compressive strength and minimal cement content. The GWWD worked with a local cement manufacturer to provide quality control services, assuring that only approved cement was used in the concrete mix. The GWWD manufactured all the concrete aggregate from two satisfactory sources of sand and gravel, located near Mile 31 and Mile 80. The GWWD also mixed the concrete aggregate before delivery to the Contractor. The work undertaken by the GWWD assured timely delivery of approved materials to the work areas. The result was a more uniform final product.

3.2.5 Geotechnical Considerations

The aqueduct traverses 157 kilometres (97.5 miles) of predominantly flat, poorly drained terrain (Figure 3-3). West of Mile 26 the aqueduct is located in the Red River valley. The soils encountered in this area are glaciolacustrine deposits consisting of clay, silt, sand and minor gravel. The deposits are predominantly silty clays ranging in thickness from 1 to 30 metres. From Mile 26 to about Mile 40 the soils are primarily glacial deposits comprised mainly of silt and sand with a discontinuous cover of glaciolacustrine clay and silt. Some discontinuous surface deposits of sand and gravel are found from about Mile 31 to Mile 39. From Mile 40 to the Intake (Mile 97.5) the terrain is predominantly covered with organic deposits such as marsh, fen, swamp and bog deposits, up to 5 metres thick, and is characterized by seasonal flooding. The organic deposits are underlain by fine grained proglacial lake and glacial till deposits ranging from clay to fine sand. Precambrian rock was encountered near the surface between Miles 70 and 78 while large Precambrian rock outcrops were encountered east of Mile 94. At nearly all locations the groundwater table is within a few feet of ground surface. (Manitoba Department of Energy and Mines, Map 81-1)

The engineers selected an alignment that would minimize excavation quantities yet provide balanced cut and fill. A variety of equipment was used for excavation including draglines, steam shovels and dredges. Comparative cost estimates showed that for typical conditions and method of payment, an average cut of 1.2 metres for the smallest aqueduct section and an average cut of 1.68 metres yielded the smallest excavation and backfilling costs per unit length of aqueduct construction. The trench excavation was completed with side slopes at 3 Vertical to 1 Horizontal (3V:1H) in what was considered to be firm soil but flattened to 1V:1H in soils that would not stand at 3V:1H (Figure 3-19). Typically the Contractor placed all excavated soils in a spoil bank located directly adjacent to the trench. The spoil bank was subsequently used as a railed for a small tramway used to deliver concrete and other supplies. The majority of the spoil bank was replaced into the trench as backfill. The practice of placing excavated soils directly adjacent to the trench resulted in numerous trench slope instabilities and also contributed to base heave of the trench floor (Figures 3-20 and 3-21). The majority of problems occurred east of Mile 85 where the depth of cut ranged from 4 to 7 metres. In this area the upper 2 to 5 metres consisted of peat underlain by fine grained soils including very soft to soft clay and loose silts.

Groundwater was a problem because nearly all of the trench excavation was completed below the water table. Groundwater control was necessary because the Specifications required that all concrete be cast under dry conditions. The Contractor was also responsible for damage resulting from the tendency of the work to float prior to filling it with water. Piping of groundwater, referred to as quicksand, into the trench occurred at numerous locations (Figure 3-22). When this condition was encountered the sand was excavated below grade between two parallel rows of wooden sheet piling and subsequently backfilled with a layer of stone and gravel (Figure 3-23). The stone and gravel layer permitted free drainage yet prevented upward migration of the underlying sand.

Groundwater seepage from the trench sidewalls was controlled by small ditches located at the toe of the trench side-walls. Seepage of groundwater from the trench bottom was controlled using a wooden box drain (Figure 3-24). The box drain was installed just below the foundation grade in a longitudinal trench along the invert centerline and backfilled with coarse stone or gravel. The box drain was left in place below the invert (Figure 3-25). The water collected from these two systems was directed to sumps and pumped out of the trench.

The aqueduct trench from Mile 85 to Mile 94 crossed an extensive peat bog ranging in depth from 2 to 5 metres. The groundwater flow from this peat layer into the aqueduct excavation resulted in numerous water related problems such as trench slope instabilities (Figure 3-26). A drainage ditch running parallel to the aqueduct trench was excavated about 90 metres north of the aqueduct trench to intercept and redirect groundwater away from the excavation. The groundwater intercepted by this ditch was redirected to the Boggy River at Mile 88 and Mile 85.

In addition to these groundwater problems, many different foundation soils ranging from peat to rock were encountered and overcome using some innovative techniques. It was necessary to provide a firm, dry foundation for the aqueduct to minimize settlement, particularly differential settlements that could crack the non-reinforced concrete. The engineers described the following soil conditions and the remedies used to overcome them.

- <u>Soupy Clay</u> – was made firm by casting rip rap sized broken stone into the clay.
- <u>Flowing clay</u> – either a wider trench was provided or the aqueduct was built with heavily reinforced invert supported on timber piles (Figure 3-27).
- <u>Quicksand</u> - sand was excavated below grade between two parallel rows of sheet piling and subsequently backfilled with a layer of stone and gravel. The stone and gravel layer permitted free drainage yet preventing migration of the underlying sand (Figures 3-22 and 3-23).
- <u>Rock Cuts</u> – foundation prepared by placing a layer of bedding sand over the rock.
- <u>Peat beds</u> – where these extended below the required invert grade, the peat was excavated to its full depth. Sand and gravel backfill was placed in the trench under water to a height of about 600 millimetres above the required invert grade (Figure 3-28). The backfill was subsequently consolidated by dewatering the trench. In some cases, a slightly thickened invert (+/- 200 millimetres) was used where the depth of the peat below the invert grade was shallow.

3.3 OPERATING HISTORY OF THE SHOAL LAKE AQUEDUCT

The engineers fully expected that the aqueduct capacity would be fully utilized within the first 25 years following completion. This expectation was based on the population growth of Winnipeg in the 40 or so years prior to aqueduct construction and the resulting population predictions made by the engineers. The estimates of the future population growth were intended to form a basis for proportioning of sizes and capacities of the future works required to meet the needs of a growing city. The engineers suggested that these predictions might be wide of the mark, when viewed backwards from the future (GWWD 1917). The demand for water and development of additional infrastructure required to utilize the full aqueduct capacity has deviated significantly from the predictions and expectations made by the engineers.

In the years prior to the aqueduct design, Winnipeg's growth was unprecedented and the city was nicknamed "the Chicago of the North." The engineers predicted that by the early 1950's Winnipeg's population would have reached the design population of 850,000 and the full aqueduct capacity would be required. Figure 3-29 shows the predicted growth of Winnipeg based on the growth trends of other major centers, such as Chicago. The population growth of Winnipeg slowed after 1915 and has yet to reach the design population of 850,000. However, average daily residential and industrial water demand usage has continued to increase. Figure 3-30 compares the original population and water demand predictions to the actual population and water demand for Winnipeg since the SLA was first commissioned in 1919.

The design capacity of 386 MLD for the Shoal Lake Aqueduct was based on the following objectives:

- 25 year design period
- Assumed average per capita consumption rate of 450 litres per day
- Design population of 850,000 people

The SLA was designed to convey water by gravity. In a pipeline conveying water by gravity flow under low heads, it is not practicable to quickly increase or decrease the flow of water in the conduit. Water demand is not constant from hour to hour, day to day or year to year. The maximum daily use can be 1.5 times greater than the average daily use for the year and for short times (1 or 2 hours) the actual demand may amount to 2.5 to 3.0 times the average daily consumption. To allow for such great fluctuations it is more economical to build reservoirs on long pipe lines than to build pipes large enough to accommodate the maximum possible demand for short times. The purpose of large reservoirs is to allow an adequate supply in response to sudden heavy demand without necessitating sudden changes in velocities in the pipeline and to store excess water not needed during hours of low demand for later use (GWWD 1917). Therefore, the SLA required a balancing storage to utilize its total capacity. Having no significant balancing storage in the system until the early 1960's meant that the SLA had to be operated continuously at a fairly constant flow rate that was not greater than the capacity of the Branch 1 Aqueduct (BR1), or about 129 MLD. This operating procedure and the minimal balancing storage also meant that the aqueduct could not be shutdown to allow for internal inspections or repairs.

From 1919 until 1960 the useable capacity of the SLA was limited by the capacity of the BR1. The full capacity of the SLA was to be developed in stages as water demand increased with the population of Winnipeg. The additional infrastructure required to develop the maximum safe capacity of the SLA, in the order of importance, is listed as follows:

- balancing storage at Mile 12.5 (Deacon Reservoir)
- booster pumping station at the Red River (Mile 3.5), Tache Booster Pumping Station
- booster pumping station at Deacon Reservoir
- second branch aqueduct (BR2) from Deacon Reservoir to future distribution pumping stations

The useable capacity of the SLA was incrementally increased as required through the addition of the above infrastructure. This is described below and shown on Figure 3-30. By 1950, water demand had reached 115 MLD and required construction of the Tache Booster Pumping Station to increase the BR1 capacity and thus the SLA capacity to 160 MLD. (Note: The designers had predicted the full capacity of the aqueduct would be required by 1950.) By 1960, demand had almost reached the BR1 capacity of 160 MLD and construction of the Branch 2 Aqueduct (BR2) was required to further increase the capacity of the SLA and to service the southern parts of Winnipeg. Two distribution reservoirs and pumping stations were added to the BR2, similar to the McPhillips distribution reservoir serviced by the BR1. The combined gravity capacity of the two branch aqueducts was about 370 MLD. However, this quantity of flow could not be realized in the SLA without balancing storage at Mile 12.5 (Deacon Reservoir). From about 1960 until 1972 the average daily demand (ADD) for water increased from 160 MLD to 200 MLD.

A flow test of the SLA was conducted in the early 1960's, which indicated that the safe operating capacity had dropped to less than 363 MLD due to slime growth on the interior surface of the aqueduct (City of Winnipeg 1965). Chlorination of water entering the aqueduct was instituted in 1966 to control slime growth and by 1967 the maximum safe capacity had been restored to 386 MLD.

Balancing storage was not provided until 1972 when the first of four storage cells (Cell 1) was constructed at the Deacon Reservoir site. This permitted the SLA to be shutdown for periods up to 5 or 6 days and also, for the first time, allowed the SLA to be operated at its maximum safe capacity of 386 MLD, when required. Three more storage cells (Cells 2, 3 and 4) were built in 1978, 1996 and 1997, respectively. The total capacity of Deacon Reservoir is 8.7 million cubic metres. This volume of water is sufficient to allow for a 28 day aqueduct shutdown while supplying the city with water at a rate equal to the yearly average daily water demand or about 230 MLD. Configuration of the Deacon Reservoir facility allows for a gravity supply to all distribution reservoirs within Winnipeg. The capacity of the branch aqueducts can be increased when required by the use of booster pumps at Deacon and/or at each of the booster pumping stations.

Few shutdowns were initiated prior to about 1990. However, since this time, numerous shutdowns have been completed to facilitate the Shoal Lake Aqueduct Condition Assessment and Rehabilitation Program, which is introduced in Section 3.4. One component of this program was to assess the buoyancy potential of the aqueduct from Mile 85.0 to the Intake at Mile 97.5. Work completed by UMA Engineering Ltd. and CH2M_Gore and Storrie determined that some sections of the SLA are at considerable risk of buoyancy when the aqueduct is dewatered. An operating policy was developed to minimize the risk of buoyancy during shutdown events. This policy required that the SLA between Mile 83 and the Intake at Mile 97.5 not be dewatered for more than 48 hours at a time and that a 'ballasting flow' of 91 MLD be restored for at least 48 hours prior to initiating another 48 hour shutdown in this area. In recent years, it has been necessary to have shutdowns greater than 48 hours in duration to facilitate inspections and repairs downstream of Mile 83. Under these extended shutdown events, the 'ballasting flow' was directed out of the aqueduct and into the Boggy River at Mile 83 of the aqueduct. This was accomplished by using stop logs and the overflow weir at this location. This practice was stopped in September 2001 after all repairs were completed to minimize the risk of aqueduct buoyancy.

At the time of writing this thesis, future plans for the water supply and distribution system for Winnipeg include construction of a water treatment plant at Mile 12.5 (Deacon Reservoir) and possibly some upgrades to the booster pumps for the two branch aqueducts. A water level telemetry system will also be added to monitor water levels in the SLA. This will become more important in the near future as the aqueduct will be operated at or near the safe maximum capacity more frequently and for longer durations.

At present, there seems to be no imminent need to secure a second potable water supply to supplement the Shoal Lake Aqueduct. Winnipeg's water pumping records for 2000 indicate the average daily demand for water is about 230 MLD with summer time peaks as high as 295 MLD and winter lows near 180 MLD. The average daily water demand has dropped since the early nineties from about 300 MLD to the present average daily water demand of about 230 MLD. This drop has been attributed mainly to improvements in toilets, shower heads and washing machines and the water conservation program called 'Slow the Flow' initiated by the City of Winnipeg in the 1990's. Based on current population trends in Winnipeg and the increasing use of water conserving devices it is estimated that the total water demand will remain fairly constant and probably not much higher than 300 to 315 MLD over the foreseeable future (Griffin, 1999).

3.4 SHOAL LAKE AQUEDUCT CONDITION ASSESSMENT AND REHABILITATION PROGRAM

During the first 46 years of service (1919 to 1964), the lack of any significant balancing storage capacity along the aqueduct precluded having any aqueduct shutdowns for the purpose of making repairs or conducting thorough internal inspections. During this time short sections of the SLA were inspected periodically using a boat placed into the aqueduct. This form of inspection did not permit more than a cursory inspection of the aqueduct. By the end of 1964, three water distribution reservoirs had been constructed within the City limits that provided enough storage capacity to allow for a two-day shutdown of the SLA. Two shutdowns were undertaken in the fall of 1964 in order to permit a thorough internal inspection of the SLA between Mile 17 and Mile 97.

In general, the 1964 internal inspection revealed that the SLA was in good condition. From Mile 19 to Mile 26, some arch cracks were observed; from Mile 26 to Mile 82, some moderate interior cracks were observed at isolated locations; and from Mile 82 to Mile 97, the only significant cracks found were in the arch sections forming a bend in the pipe at Mile 85.65. Groundwater was found to be infiltrating into the pipe through a significant number of construction joints (City of Winnipeg 1965).

In 1987, an internal investigation revealed that several sections of the SLA required major repairs. Although repairs to the aqueduct were on-going from year to year the 70 year old aqueduct was due for a thorough condition assessment. This work was necessary to evaluate the condition of and maintain Winnipeg's only water supply. Early in the 1990's, the original condition assessment program grew into a comprehensive condition assessment and rehabilitation program. This program included the aqueduct and all of its pertinent structures. All engineering work and repairs will be completed by the end of 2003.

The objective of the condition assessment was to determine the general condition of the aqueduct and whether it could provide a continuous and reliable supply of water for at least more 50 years. The condition assessment program revealed that the majority of the aqueduct was in remarkably good condition but there were some areas of the aqueduct requiring attention. The condition assessment program considered the following performance characteristics (CH2M Gore and Storrie, 1998):

- Structural Performance Considerations: all issues related to loads acting on the aqueduct structure, including hydrostatic pressures (buoyancy), stability of the structure when cracked, infiltration and exfiltration of water through cracks.
- Environmental Performance Considerations: all issues related to the ability of the concrete to resist attack caused by environmental conditions such as chemical attack (for example, sulphate) and temperature changes (frost).
- Hydraulic and Operational Performance Considerations: maximum safe capacity of the aqueduct, hydraulic restrictions, overflow capacities.
- Safety/Vulnerability Performance Considerations: issues relating to a physical rupture of the aqueduct caused by structural failure and/or sabotage.

The activities required to assess these above performance considerations are too numerous to list here but included: internal inspections, non-destructive testing, concrete core sampling, monitoring of cracked sections to determine behavioural characteristics, soil sampling, surveys, flow tests and analysis (CH2M Gore and Storrie, 1998).

This thesis has been prepared to examine issues associated with just one of these characteristics, specifically the structural performance consideration of buoyancy. The engineers who designed the aqueduct were aware of the buoyancy potential of the aqueduct and they made assumptions regarding the conditions required for buoyancy to develop (Section 3.2.2). As mentioned in Section 3.2.2, the engineers accepted some risks in the design as it was not financially feasible for a young community like Winnipeg to build the aqueduct on the basis of no risk. Recent condition assessments (Gore and Storrie, 1995) have confirmed the buoyancy potential of the aqueduct.

Chapter 4 reviews the Buoyancy Assessment Program and presents work done by the author in the framework of risk management. This is a relatively new approach in civil engineering and it is believed that this is the first time it has been used in such a structured way in a major geotechnical project in Manitoba.

4.0 BUOYANCY ASSESSMENT PROGRAM

The Buoyancy Assessment Program, a component of the Shoal Lake Aqueduct Condition Assessment and Rehabilitation Program (SLA_CARP), was undertaken to determine whether or not the stretch of aqueduct from Mile 85.0 to the Intake (Mile 97.5) would be at risk of flotation when dewatered for extended periods of time. It was also to provide recommendations for safeguarding the aqueduct against buoyancy failures. The work undertaken to complete the assessment included a review of existing information, an evaluation of soil and groundwater conditions, surveys, internal aqueduct investigations, evaluation of different rehabilitation options, and a buoyancy analysis.

Buoyancy of the aqueduct is the ultimate limit state (ULS) that is the principal focus of this thesis. There are other limit states that had to be evaluated based on the remedial option (granular ballasting) selected to reduce the risk of buoyancy. The ultimate limit states associated with granular ballasting include bearing capacity, and structural collapse. The serviceability limit states include settlement and cracking. These additional limit states will be discussed in following chapters.

4.1 BUOYANCY CONCERN

The engineers who designed the SLA identified buoyancy as a concern and believed this was addressed during design and construction. The issue of buoyancy re-surfaced in the early stages of the SLA_CARP because the aqueduct would have to be shutdown for extended periods of time (greater than 48 hours) to facilitate internal inspections and repairs.

Buoyancy is the tendency of an object to float in a fluid. Buoyancy of the aqueduct is only of concern during shutdowns when there is no water flowing in the aqueduct. At all other times the minimum flow rate of about 125 MLD provides enough additional weight required to protect the aqueduct from buoyancy. In order to complete the SLA_CARP numerous shutdowns were required. The City of Winnipeg's Water and Waste Department also desired to have the operational flexibility to shutdown the aqueduct at a moment's notice in the event of an emergency and not have to worry about buoyancy of the aqueduct. Re-assessment of the potential of the aqueduct to become buoyant was warranted because the aqueduct is the only source of potable water for Winnipeg and because the operating requirements of the aqueduct are significantly different from what was originally anticipated by the design engineers. In addition, review of original design documents revealed that the designers made assumptions, not necessarily conservative, regarding the specific conditions required for buoyancy to occur.

As discussed in Section 3.3, an interim operating policy was instituted to reduce the buoyancy potential of the aqueduct between Mile 85.0 and the Intake at Mile 97.5 until the Buoyancy Assessment Program and the required remedial repairs were completed. This operating policy did not allow the aqueduct upstream of Mile 83 to be completely dewatered for more than 48 hours.

4.2 EXISTING INFORMATION

A significant amount of archived information was available to assist in understanding the buoyancy problem from the perspective of the original design engineers. The most valuable information includes drawings of the various aqueduct sections, a record drawing indicating the aqueduct section and invert types constructed east of Mile 85 (GWWD, 1919), three technical papers summarizing the design process, construction difficulties and operating guidelines (Chase, 1920a and 1920b and Feurtes, 1920) and a soil profile drawing (Figure 4-1) showing the soil types encountered during excavation of the aqueduct trench. Construction photographs also helped to understand the conditions encountered during construction. This information was also used to (1) help locate stretches of aqueduct having the highest risk of flotation, (2) provide critical information used in the buoyancy analysis and (3) determine what additional information was needed to complete the buoyancy analysis.

During the design phase for the aqueduct, the engineers recognized the buoyancy potential of the aqueduct between Mile 85.0 and the Intake at Mile 97.5 based on the soil and groundwater conditions encountered during the aqueduct route selection investigation. Consideration was given to hauling in granular backfill to provide the necessary resistance but the most economical plan was a special thickened, or 'gravity invert' (Figure 3-15). This invert was designed to:

> *"secure stability and permanence for the aqueduct, full and empty, when the groundwater level was above the arch of the aqueduct and the backfill so soft, light and lacking cohesive properties as to be of practically no value except for frost-proofing"* (Feurtes, 1920).

The engineers recommended that the gravity invert only be constructed at locations where the three following conditions occurred (Feurtes, 1920):

- groundwater levels above the aqueduct (completely submerged)
- backfill is of light weight (organic in nature)
- porous foundation soil

The total length of aqueduct constructed with the 'gravity invert' is about 340 metres. The locations between Mile 85.0 and the Intake at Mile 97.5 where the 'gravity invert' was constructed are shown on Figure 4-2. These locations were also mentioned in an operating manual for the aqueduct as follows:

> *"In certain stretches of the work along Snake Lake and at Mile 87 where the uncontrollable ground water surface stands above the conduit and where the material available for the refill was of small weight per cubic foot, the conduit has been made safe from flotation by weighting the invert. In these places the invert is two feet or more in thickness; these weighted portions are not continuous, particularly along Snake Lake, but are built where the sub-soil seemed to be of such a nature as to permit sufficient percolation to establish at some time an upward pressure sufficient to cause the aqueduct when empty to rise and float. The experience to date indicates that all this aqueduct is safe"* (Chase, 1920b).

The aqueduct had been in continuous service for about 1 year when this comment was made. Interestingly, the flow rate used during this period was similar in magnitude to the ballasting flow rate of 91 MLD (Section 3.3) used in the 1990's to reduce the risk of buoyancy during extended shutdowns.

Information needed to complete the Buoyancy Assessment Program included: backfill types and unit weights, concrete unit weights, aqueduct dimensions, foundation soil types and modern engineering properties, groundwater levels.

4.3 IMPACT OF ORIGINAL DESIGN ASSUMPTIONS REGARDING BUOYANCY

As mentioned in the previous section, only 340 metres of 'gravity invert' were constructed between Mile 85.0 and the Intake at Mile 97.5, an area that is 20,000 metres (12.5 miles) long. The 'gravity inverts' represent less than two percent of the length. Discontinuous use of the gravity invert may be explained by the following statements:

> "*to have stood upon the principle of 'no risk' in the designs would have rendered the project totally impossible, on account of the great cost involved.*" (Fuertes, 1920).
>
> "*The expense of such an invert was very great and was avoided wherever the earth of the trench floor seemed fairly tight.*" (Chase, 1920a).

It appears that the aqueduct design may not be conservative when the existing soil and groundwater conditions between Mile 85.0 and the Intake at Mile 97.5 are compared to the three conditions required for the use of a 'gravity invert' (Section 4.2). The designers may not have envisioned the aqueduct being dewatered for three to four weeks at a time. Upon completion of all remedial repairs that address the concern of buoyancy from Mile 85.0 to the Intake (Mile 97.5) the aqueduct in this area will be shutdown for periods of time up to 4 weeks in duration. (note: these repairs were completed in May 2001 and the aqueduct was shutdown without incident for a period of about 3 weeks in the fall of 2001.). The following discussion reviews the original three conditions required for buoyancy to occur and the existing conditions.

The first condition required that the aqueduct be submerged. Simple analysis shows that if the aqueduct, with no backfill, is about sixty percent submerged, as defined in Figure 4-3, the aqueduct self-weight is approximately equal to the buoyant force. (Note: A full discussion on buoyancy is presented in Section 4.4. Table 4-1 summarizes the self-weight and buoyant forces when the pipe is partially submerged (sixty percent). Uplift resistance due to the backfill has been ignored in this simple analysis. Excluding the backfill is not unreasonable because the submerged unit weight of organic soils is very low (typically less than 2 kN/m^3) and shearing resistance is unreliable at best due to the large strains that would be required to mobilize shear strength. This shows that any portion of the aqueduct that is submerged greater than sixty percent may be at risk to buoyancy, depending on the nature of the backfill. Under current conditions approximately ninety percent (11.25 miles) of the aqueduct from Mile 85.0 to the Intake at Mile 97.5 is fully (100 percent) submerged, including the backfill directly over the pipe.

The second condition required that the backfill be organic (light weight). Between Mile 85.0 and the Intake at Mile 97.5 about 40 percent (5 miles) of the pipe has organic (light weight) backfill soils, about 43 percent (5.5 mile) of the aqueduct has a mixture of organic and mineral backfill soils and only 17 percent (2 miles) of the aqueduct has backfill soils comprised of more than 90 percent mineral soils. The general locations of mineral and organic soils can be delineated from the soil profile shown on Figure 4-1.

The third condition requires that a porous foundation soil be present. This condition was based on field observations during construction and it is not known what soil types, other than sand and gravel, were considered porous. Since the first two conditions required for buoyancy generally exist east of Mile 85.0 the third condition appears to be the overriding factor in determining whether or not a 'gravity invert' was constructed. Based on test holes drilled in the area, the foundation soils typically range from sandy silts to silty clays. Approximately forty-five percent of the aqueduct between Mile 85.0 and the Intake at Mile 97.5 has a silt foundation and the remainder has a clay foundation.

Two things that the design engineers may have overlooked is the time-dependent nature of groundwater flow in fine grained soils and the potential influence of the box drain located directly below the aqueduct invert (see Section 3.2.5). During a shutdown in June 1998 several core holes were drilled through the invert slab near Mile 94.6 and

Mile 95.7. Both locations showed considerable seepage from the underlying foundation soil. Gravity inverts had not been constructed at these locations. Due to dewatering efforts used during construction to keep the excavation dry until all concrete had cured, current groundwater and seepage conditions may be different than those observed during construction. Some pertinent comments from the Chief Engineer (W.G. Chase) for the Greater Winnipeg Water District (GWWD) are included below:

> *"Before the pouring of invert concrete it was required of the contractors that they furnish a dry foundation; this was obtained by means of side ditches in the trench bottom which collected the water seeping in through the walls of the trench and by use of a central longitudinal box drain laid in gravel; this box drain,* Figures 3-24 and 3-25, *was temporarily outletted to sumps from which the water was continuously pumped during construction. These outlets were plugged with backfill and it is believed that in time the box drain itself may be choked with earth, though at the time of completion of the aqueduct it was noted at several points that the box drain was free for miles above these points, and that the water therein was under a considerable pressure head; this pressure head can at no time be greater than that corresponding to the elevation of the adjacent ground waters, unless the sub-soils should be so tight as to prevent percolation from the box drain; it is not likely that the upward pressure from this pressure within the box drain can or will do any injury to the conduit"* (Chase, 1920b).

Investigations undertaken during the course of the Buoyancy Assessment Program revealed that the box drains are still hydraulically connected to the groundwater surrounding the aqueduct. These box drains may now function in a negative manner by supplying the source of water necessary to generate the uplift pressures required for buoyancy to develop. Uplift pressure generation in fine grained soils such as silts and especially clays is a time-dependent factor that could become a concern during extended aqueduct shutdowns (3 to 4 weeks).

A thorough review of the aqueduct buoyancy potential was warranted in light of (1) the assumptions made by the designing engineers regarding the conditions for buoyancy to develop, (2) the current aqueduct operating requirements, and (3) the great importance of this piece of infrastructure.

One additional consideration that had to be addressed during the Buoyancy Assessment Program was the strength of the non-reinforced invert slabs shown in Figures 3-10, 3-11 and 3-14. The hydrostatic or uplift pressure acting on the underside of the invert not only contributes to the buoyant force but generates stress in the invert slab. The hydrostatic pressure acting upwards on the invert slab is directly proportional to the height of water above the invert. As discussed in Section 3.2.2, five main invert sections were used during construction of the aqueduct, two were reinforced with steel and the remaining three were not. A full discussion of this issue is beyond the scope of this thesis. However, a brief summary is included because the type of repairs used to reduce the risk of buoyancy was impacted by invert strength.

The nature of the invert stresses is variable, depending on how much lateral resistance is available to prevent spreading of the arch legs. For example, the non-reinforced invert slabs will go into pure compression if sufficient lateral resistance is available. If lateral resistance is provided by a highly compressible backfill, such as organics, non-reinforced invert slabs will go into bending and the tensile stresses will develop in upper portions of the invert slab. If the tensile strength of the concrete is exceeded, longitudinal cracks will develop along centerline of the invert. In extreme cases the invert could fail.

A finite element analysis of the aqueduct structure was completed by CH2M_Gore and Storrie Ltd. The results revealed that the 'R' and 'B' aqueduct sections, for all invert types, have enough strength to resist cracking under the range of hydrostatic uplift pressures expected. The most critical aqueduct section was the 'S' aqueduct section constructed with the 'E' style invert section shown in Figure 3-14. The 'S' section is the largest of all the sections used and measures 3.277 metres wide. The finite element analysis of the 'E' section invert revealed a stress concentration where the curved invert bottom abruptly becomes flat or horizontal, this feature is visible on Figure 3-14. The stresses that would be induced at this location in the invert due to additional backfill load (granular ballast) could be large enough to break the invert slab. The risk associated with placing granular ballast on the 'SE' aqueduct section was too high. The 'SE' section type was mainly used east of Mile 93.5. However, 70 metres of aqueduct at Mile 91.7 was constructed with the 'SE' section. The invert at this location was strengthened prior to completing granular ballasting to address buoyancy concerns. East of Mile 93.5, buoyancy concerns were addressed with static water ballasting. Static water ballasting will be discussed more fully in Section 4.10.

4.4 AQUEDUCT BUOYANCY CONCEPT

Buoyancy of the aqueduct is only of concern when the aqueduct is shutdown and there is no water flowing in the pipe. The weight of water in the pipe under the minimum flow rate of 125 MLD provides more than enough additional weight to prevent buoyancy. The buoyant force is generated by groundwater pressures acting on the underside of the aqueduct. The buoyant force is resisted by the weight of the aqueduct and any water it contains whether static or flowing, the weight of overlying backfill, and the shearing resistance of backfill soils. A buoyancy concept was developed to help determine what information would be required for a buoyancy model.

The disturbing (buoyant) force is the net sum (integral) of all water pressures acting on the pipe. A simplified model showing the buoyant force and the resisting forces is shown on the force diagram in Figure 4-4. The aqueduct is stable when the resisting forces are equal to or greater than the buoyant force. The buoyant force is calculated using Archimedes principle, which states that the buoyant force is equal to the weight of fluid (water in this case) displaced by a body (the aqueduct). The use of Archimedes principle assumes static groundwater conditions, that is, no upward or downward gradients. The buoyant force always acts vertically upwards. The magnitude of the buoyant force is a function of the degree to which the aqueduct is submerged. For example, if the groundwater surface is coincident with the underside of the aqueduct or lower, the buoyant force is zero. At approximately 60 percent submergence, as shown in Figure 4-3, the buoyant force is approximately equal to the weight of the aqueduct. If the groundwater surface is coincident with or higher than the top of the aqueduct (100 percent submerged) then the maximum buoyant force is realized. The change in buoyant force with increasing degrees of submergence is illustrated on Figure 4-5. The groundwater levels between Mile 85.0 and the Intake at Mile 97.5 are generally near the top of the backfill berm. That is, the aqueduct is 100 percent submerged.

In fine grained soils such as clay, development of the buoyant force is time-dependent. At least 50 percent of the aqueduct between Mile 85.0 and the Intake at Mile 97.5 is founded on clay. The time-dependent nature of porewater response was ignored in the aqueduct buoyancy model for several reasons. One, the time-dependent nature is complex and difficult to model. Two, the presence of relatively highly permeable soil layers (for example, sand seams) just below the foundation level would produce uplift pressures acting on the overlying clay layer and hence the aqueduct. Given the length of aqueduct under consideration (12.5 miles) it was not feasible to investigate the likelihood of this occurring. Three, the presence of the box drain immediately below the aqueduct invert slab could provide the source of water needed to generate the uplift pressures. Four, given the importance of the aqueduct to Winnipeg, the risk associated with including this time-dependent factor in the buoyancy model was seen as being too great. The costs associated with repairing one 14 metre long aqueduct section that failed as a result of buoyancy would go a long way towards providing additional resisting forces to prevent buoyancy. In addition, if one aqueduct segment failed due to buoyancy, it is likely that others could fail and remedial repairs to prevent buoyancy would have to be completed.

As outlined earlier, the forces resisting the buoyant force include:

- self-weight of the aqueduct
- weight of backfill
- backfill shear resistance along the failure surfaces

The self-weight of the aqueduct was determined using the design dimensions for each aqueduct section type (Table 3-1) and the unit weight of concrete based on measured values (Section 4.8.3). The dimensions and areas required for the buoyancy analysis of each aqueduct section type constructed between Mile 85.0 and the Intake at Mile 97.5 are included in Table 3-1. The buoyant force is calculated from the volume of water displaced by the aqueduct. Any changes in total volume of the aqueduct section result in proportional changes in buoyant force and also self-weight. For example, if the internal dimensions remain constant but the outer envelope increases (thicker concrete) the buoyant force will increase. However, the incremental increase in self-weight will be about 2.4 times greater than the incremental increase in buoyant force because the unit weight of concrete is about 2.4 times heavier than water. This results in a more stable structure. In contrast, if the outer envelope decreases in size (thinner concrete) the decrease in buoyant force is about 2.4 times smaller than the decrease in self-weight. On the other hand, if the outside shell of the aqueduct remains constant but the internal dimensions change (increased or decreased concrete thickness) the buoyant force will remain constant and only the aqueduct self-weight will increase or decrease proportionally with a change in concrete thickness.

Steel forms, Figure 3-17, were used to cast the arch. Given the way in which the forms were assembled it is expected that the cross-sections from any given set of forms were fairly uniform. The largest differences in cross-section would likely have resulted from differences between sets of forms. It is not known if the inner and outer forms were used as matched sets or if they were intermixed, which could account for some variability in cross-section. The invert section dimensions were controlled using wooden templates to ensure proper thickness and shape. It is expected that the tolerances were fairly rigid because the internal dimensions and concrete thickness are very important for hydraulic capacity and structural capacity, respectively.

The buoyancy assessment work done at UMA Engineering Ltd. (1994 to 1999) assumed that the aqueduct was built to the design dimensions. Additional work done for this thesis includes determining the effect of varying dimensions of the aqueduct section and incorporating this in to the buoyancy analysis presented in Chapter 5.

As shown in Figure 4-4, the assumed model for the backfill weight was divided into two components, the crown backfill and the arch leg backfill. The arch leg backfill volume is essentially constant for each aqueduct section type while the crown backfill volume varies with backfill depth above the crown. The width of the backfill zone contributing to stability of the aqueduct is assumed to be no wider than the aqueduct at its widest point. Crown backfill depths were determined using the results from a profile survey of the backfill berm (Figure 4-2).

The existing backfill soils can be categorized as either mineral (clay, silts, fine sands) or organic soil. Except for a few isolated areas, the backfill is comprised of the soil removed from the trench during excavation. Figure 4-1 shows the soil profile encountered during excavation of the aqueduct trench. The weight of the backfill varies with the degree of submergence. However, in most cases the backfill is nearly 100 percent submerged. Submerged unit weights were used to calculate the weight of the arch leg backfill. Submerged or bulk unit weights were used to calculate the weight of the crown backfill, depending on groundwater levels.

Backfill shear resistance depends on the shape of the failure surface and on the type of backfill soil. Several possibilities exist for the shape of the failure surface. A vertical failure surface is an appropriate assumption for remoulded cohesive soil such as soft clays while sloped or curved failure surfaces are more typical for granular soils (Vesic, 1971). A vertical failure surface extending upward from the widest point of each aqueduct section type (Figure 4-4) was used in the buoyancy analysis because the majority the backfill soils encountered are expected to behave more like a soft cohesive material rather than a granular soil.

Organic (peat) backfill soils typically have low shear resistance and require large (unacceptable) shear displacements to generate any shear resistance. Therefore, only mineral soils have been considered when calculating shear resistance. Cohesive soils (clays) provide shearing resistance through cohesion and while coarser soils (silts and sands) provide shearing resistance through friction.

The cohesive component of shear resistance was calculated using measured undrained shear strengths of the clay backfill applied over the thickness (or height) of the clay backfill above the outer edge of the invert. The height of the clay backfill contributing to shear resistance varied along the length of the aqueduct due the type of soils excavated from the aqueduct trench (Figure 4-1). Except for a few isolated areas, the material used for backfill was the excavated material. That is, in areas where the only backfill available was organic, no clay backfill was available to be placed in the trench. In areas where the excavated soil was about 50/50 organics/clay it was assumed that a compacted backfill layer of clay, 1.0 to 1.2 metres thick, was preferentially placed at the bottom of the backfill zone (Section 3.2.3 and Figure 3-18). This was verified by drilling test holes in the backfill soils to determine the soil types (Section 4.5.2). In areas where the backfill was predominantly clay, the height of backfill was determined from backfill test holes drilled by UMA Engineering Ltd. (Section 4.5.2).

The frictional shear resistance of silt and sand backfill was determined using the frictional component of Coulomb's shear strength equation. The normal force acting on the failure planes is the lateral at-rest earth pressure calculated using earth pressure theory. The angle of internal friction for the soil was estimated using an empirical correlation between Standard Penetration Testing results and angles of internal friction, Equation 4-1 (based on charts found in Bowles, 1968).

[4-1] $$\phi = 0.28N + 27.4$$

Where:

ϕ - Angle of Internal Friction (degrees)
N - SPT Blow Count (blows per 300 millimetres)

The normal force acting on the failure planes was calculated using Equation 4-2:

[4-2] $$P = (0.5)(K_o)(\gamma')(H^2)$$

Where:

P - normal force (kN)
K_o - lateral at-rest earth pressure coefficient
γ' - submerged unit weight (kN/m^3)
H - height of mineral backfill

A K_o value of 1.0 was assumed for the buoyancy analysis largely due to the nature and consistency of the backfill soils (UMA Engineering, 2000).

The Buoyancy Assessment Program has led to the following general notes about buoyancy of the aqueduct.

- Where the backfill consists entirely of mineral soils, sufficient resisting forces may be available to prevent buoyancy even when the aqueduct and the backfill berm are entirely (100%) submerged.
- Where the backfill consists entirely of organic (peat) material, the pipe may be at risk to flotation even though the aqueduct is not completely submerged.

A detailed discussion of the buoyancy analysis is included in Chapter 5.

4.5 FIELD PROGRAM

The field program consisted of four general components to gather information needed to complete a buoyancy assessment of the aqueduct between Mile 85.0 and the Intake at Mile 97.5. The work was completed over a period of about 5 years between 1994 and 1999. In general, the work included (1) drilling test holes to collect soil and groundwater information, (2) performing internal inspections of the aqueduct to collect information about its construction, and (3) topographical surveying to determine backfill and invert profiles, tie in test holes and measure groundwater levels.

4.5.1 Detailed Test Holes

Following a review of existing geotechnical data, construction records and construction photographs twenty-one locations were targeted for detailed test holes between Mile 85.0 and Mile 96.7 (Figures 4-1 and 4-2). The locations represent a variety of subsurface conditions, ranging from soft clay to sandy soils of variable thickness. Test holes were also targeted in areas where construction problems such as heaving of the excavation base (Figure 3-20) or side slope instabilities (Figure 3-21) were known to have occurred.

Test holes were advanced to auger refusal or to a maximum depth of 21 metres. Depths in excess of 21 metres were probed by either hydraulically pushing or driving a Standard Dutch Cone with a 65 kilogram hammer. Probing with the Dutch Cone was limited to a maximum test hole depth of 26.5 metres or until refusal was reached on suspected bedrock.

Test holes were logged visually during drilling. Undisturbed (Shelby tube) samples were taken in cohesive soils at regular intervals. Disturbed (split spoon) samples were recovered during Standard Penetration Testing (SPT), conducted primarily in non-cohesive sands and silts. All samples were protected to prevent moisture loss and freezing and transported to the Soil Laboratory at UMA Engineering Ltd. in Winnipeg. Standpipe and vibrating wire piezometers were installed at selected locations to measure groundwater levels. Test hole logs that are representative of the general soil and groundwater conditions between Mile 85.0 and the Intake at Mile 97.5 are included in Appendix 4-1. Groundwater levels measured in the piezometers are shown on Figure 4-2.

4.5.2 Backfill Test Holes

Approximately two hundred and seventy (270) backfill test holes were drilled along the aqueduct centerline and just adjacent to the aqueduct. In the following sections, the test holes drilled on centerline are referred to as crown backfill test holes and the test holes adjacent to the aqueduct are referred to as arch leg backfill test holes. Figure 4-6 shows the relative locations of the test holes at a typical section. The crown backfill test holes were advanced to the crown of the aqueduct and the arch leg backfill test holes were advanced to the foundation level.

Soil and groundwater conditions at each test hole location were logged visually, and representative (auger cuttings) samples were recovered and preserved to prevent moisture loss. At selected locations, undisturbed soil samples were recovered using a thin walled acrylic piston sampler and backfill unit weights were measured directly in the field. All samples were protected to prevent moisture loss and transported to the Soil Laboratory at UMA Engineering Limited in Winnipeg for further testing.

4.5.3 Aqueduct Shutdowns

Approximately ten shutdowns were scheduled to permit access inside the aqueduct for collecting information. Because of buoyancy concerns, each of these shutdowns was limited in duration to a maximum of forty-eight (48) hours (Section 3.3). Due to the length of time required for dewatering the available work window inside the aqueduct was limited to about 36 hours.

A number of tasks were completed during the shutdowns. These included:

- collection of concrete cores for unit weight and thickness measurements, and strength testing;
- collection of backfill soil samples directly adjacent to the arch
- collection of foundation soil samples and undrained shear strengths from just below the invert;
- verifying the presence (or absence) of the box drain below the invert and measuring the hydraulic head in the box drain;
- measuring internal dimensions of the aqueduct.

Thirty concrete cores were taken from the arch and the invert. The cores were shipped to UMA Engineering Ltd. for photographing and to measure unit weights and core dimensions. Fourteen of the cores were shipped to the University of Alberta for strength testing under the guidance of Dr. Dave Rogowsky, P.Eng. who had previously been involved in the aqueduct project while employed at UMA Engineering Ltd.

Where possible, shallow test holes were drilled into the foundation soils (12 test holes) and backfill soils (5 test holes) to evaluate soil and groundwater conditions. Undrained shear strengths of the foundation soils were measured insitu with a field vane.

Small diameter (25 millimetres) holes were drilled through the invert slab into the wooden box drain on centerline. Steel grout injection ports fitted with valves were installed in each hole to facilitate measurement of hydraulic head in the box drain.

Internal dimensions of individual aqueduct segments were measured about every 160 metres. Each aqueduct segment was measured in at least one location. Additional measurements were taken if the measurements varied significantly from the design measurements. A total of 266 segments were measured between Mile 85.0 and the Intake at Mile 97.5. This work was originally completed to provide data for hydraulic modeling of the aqueduct. This information was not used in the buoyancy assessment work completed by UMA Engineering Ltd. but was incorporated into the subsequent work done specifically for this thesis.

4.5.4 Surveys

Several surveys were conducted by the City of Winnipeg, Pollock and Wright and UMA Engineering Limited to tie in test hole locations, measure groundwater levels and to determine profiles of the backfill berm and the aqueduct invert.

4.6 SITE CONDITIONS

As the Buoyancy Assessment Program progressed it became obvious that drainage and backfill characteristics defined three distinctly different stretches between Mile 85.0 and the Intake at Mile 97.5. These three stretches are listed below and delineated on Figures 4-1 and 4-2:

- Boggy River Stretch – Mile 85.0 to Mile 88.5
- Summit Stretch – Mile 88.5 to Mile 93.5
- Snake Lake Stretch – Mile 93.5 to the Intake at Mile 97.5

These three stretches have unique drainage characteristics that impact the aqueduct from a buoyancy perspective and an invert stress perspective (Section 4.3). The drainage and backfill characteristics for each stretch are defined below.

<u>Boggy River Stretch (Mile 85.0 to Mile 88.5)</u>

The east and west extremities of the Boggy River Stretch are fairly well drained but are joined by a poorly drained peat bog from about Mile 85.8 to Mile 87.8. Groundwater levels within this peat bog are typically at or near ground surface. West of Mile 85.5 and from Mile 87.8 to Mile 88.5 the Right-of-Way is relatively well drained but prone to flooding when the nearby Boggy River overflows into the aqueduct Right-of-Way. A hydrology study of the Boggy River in this area helped estimate the maximum groundwater elevation during flood conditions (UNIES, 1998).

Except for two small areas between Mile 85.0 and Mile 85.5 and from Mile 87.8 to Mile 88.5 the aqueduct and the crown backfill in this area are completely submerged (Figure 4-7). The backfill in this stretch is predominantly organic in nature with occasional pockets of fine grained mineral soils such as clays and silts (Figure 4-1).

The Boggy River Stretch is linked to the Summit Stretch by the drainage ditches on either side of the aqueduct that run from Mile 88.0 to Mile 93.5. These two ditches ultimately drain to the Boggy River via the drainage siphon and offtake ditch at Mile 87.99.

<u>Summit Stretch (Mile 88.5 to Mile 93.5)</u>

As shown on Figure 4-1, a natural high point in the terrain occurs near Mile 93.5. This high point is also a watershed boundary. To minimize the depth of cut for the aqueduct trench a very shallow grade (0.0011 percent) was used between Mile 91.15 and the Intake at Mile 97.5. Even with this shallow grade excavation depths up to 7 metres were required. To minimize the stresses in the aqueduct structure due to backfill weight, the excavation was not backfilled to the original ground surface. Ditches were subsequently constructed on the north and south sides of the aqueduct. These ditches drain in a westerly direction to an offtake ditch at Mile 87.99. Water levels in these ditches are strongly influenced by beaver dams. If the dams are not removed on a regular basis they can impound water anywhere from 0.6 to 0.9 metres above the crown backfill berm.

The aqueduct and a high percentage of the crown backfill in this area are completely submerged (Figure 4-8). The backfill in the Summit Stretch is generally a mixture of organic and fine grained mineral soils such as clays and silts (Figure 4-1).

Snake Lake Stretch (Mile 93.5 to Intake at Mile 97.5)

Drainage in this stretch is typically poor, with water levels mainly controlled by the height of the aqueduct backfill berm and/or levels in the nearby Snake Lake. East of Falcon River (Mile 96.7), water levels within the aqueduct Right-of-Way reflect the levels in Shoal Lake.

The watershed boundary located near Mile 93.6 prevents Snake Lake from draining west to the Boggy River. This natural watershed boundary was breached during excavation of the aqueduct trench and was only partially restored when the aqueduct trench was backfilled. This was done to reduce stress levels in the aqueduct structure due to the backfill. An earth dam about 60 metres long was built across the full width of the aqueduct trench near Mile 93.6 at an elevation of 324.1 meters. This elevation represents the expected high water elevation in Snake Lake, based on available data in 1918. The dam elevation is about 2 metres lower than the surrounding ground surface. UNIES (1999) prepared a hydrologic study for Snake Lake to help determine flood elevations for this area.

The aqueduct and the overlying crown backfill in this area are completely submerged (Figure 4-9). The backfill within this stretch ranges from either organic or mineral soils to a combination of organic and mineral soils (Figure 4-1).

4.7 SUBSURFACE CONDITIONS

4.7.1 Regional Geology

Manitoba Department of Energy and Mines has described the regional geology and history of glaciation for the area east of Mile 85.0 (Manitoba Department of Energy and Mines, Map 81-1). The region is in Precambrian terrain covered with up to 6 metres of post glacial organic deposits (swamps and bogs). The organic layer is underlain by fine textured lacustrine (silts and clays) and glacial (till) deposits ranging in thickness from less than 1 metre to greater than 18 metres.

4.7.2 Soil Profile

The general profile of the subsurface (foundation) and backfill soils are described separately in the following sections. Engineering properties of the various soil units are presented in Section 4.8.

4.7.2.1 Subsurface (foundation) soils

West of Mile 93.5, the subsurface soils generally consist of soft to firm silty clays of low to high plasticity and loose silts. East of Mile 93.5, the subsurface soils consist of loose silts and sands. Depth to bedrock is variable, ranging from outcrop at Mile 93.5 and Mile 95.3 to greater than 26 metres west of Mile 93.5. There is generally good agreement between the shallow foundation soils encountered in the detailed test holes and the soil profile shown on Figure 4-1.

4.7.2.2 Backfill Material

The backfill ranges from organic (peat) to mineral soils such as clay, silt and sand and combinations thereof. In many cases the soil types are layered or intermixed. The backfill soils have been described in Section 4.6.

The original Construction Specification (GWWD 1914) for backfill required that select earth be placed by hand and compacted in the base of the trench to a depth of 1.2 metres above the invert (Section 3.2.3 and Figure 3-18). This zone shall be referred to as the compacted backfill zone. Due to the cost of imported backfill material, only material taken from the excavation was used for backfilling with mineral soils being placed preferentially in the base of the excavation. Imported backfill (sand and gravel) was only used where backfill quantities from the excavation were not sufficient to provide adequate cover.

4.8 LABORATORY AND FIELD TESTING RESULTS

4.8.1 Foundation Soils

Soil samples from the detailed test holes were transported to the Soils Laboratory at UMA Engineering Ltd. for further classification and testing. Tests performed include the determination of water content, plasticity (Atterberg limits), unit weight, grain size distribution, undrained shear strength and compressibility (consolidation testing). The scope of this thesis does not warrant a detailed discussion of these test results, although the important findings are summarized below. Typical test results for the foundation soils are included on the test hole logs presented in Appendix 4-1.

4.8.2 Backfill Soils

Samples of the backfill soils were transported to the Soils Laboratory at UMA Engineering Ltd. to determine water contents and unit weights. The water contents averaged 134 percent and ranged from 23 percent in silt backfill to 550 percent in organic backfill. This range of values is too high to be useful in statistical analysis. It will be shown later in this chapter and Chapter 5 how the data was interpreted for use in the buoyancy analysis.

Unit weights of the backfill soils were determined from undisturbed (Shelby tube) samples recovered from the detailed test holes and from piston tube samples collected from the backfill test holes. The samples were assumed to be fully saturated because all samples were collected below the water table. From forty-eight samples, the saturated unit weight ranged from 9.8 to 19.0 kN/m^3 with an average of 13.0 kN/m^3. Figure 4-10 shows a histogram of the measured backfill unit weights. Organic soils tend toward the lower end and mineral soils (clay, silt, sand) correspond to the higher end of the saturated unit weight range, respectively. The soils in the middle range represent a matrix of intermixed organic and mineral soils.

The saturated and dry unit weights were plotted against sample water content to determine if a reliable method of estimating backfill unit weight could be developed based on water content, thus reducing the reliance on detailed sampling and laboratory testing. The results are shown on Figure 4-11. Non-linear regression analysis methods were used to fit a power model to the data. The model was used to estimate dry unit weights from water content measurements. The dry unit weights were converted to saturated and submerged unit weights for use in the buoyancy analysis (Chapter 5). The goodness-of-fit statistics for the power model are: correlation coefficient, $R^2 = 1.15$; standard error of estimate = 0.91 kN/m^3. Obviously an R^2 greater than 1.0 is not possible. Because these fit statistics were calculated using a transformation of the criterion variables, the standard error of estimate is a better measure of accuracy (Ayyub and McCuen, 1997). Ayubb and McCuen state that if the standard error of estimate is considerably less than the standard deviation and is near zero, the regression analysis has improved the reliability of prediction. The standard deviation for the data set is 4.7 kN/m^3. Since the standard error (0.91 kN/m^3) is only about 19% of the standard deviation, it can be reasonably concluded that the regression analysis has improved the reliability of prediction.

Undrained shear strengths were estimated at the ends of Shelby tube samples using a pocket penetrometer, torvane and a laboratory vane. Field vane tests were used to measure insitu undrained shear strengths in hand auger holes. Undrained shear strengths of the clay and silty clay backfill range from 10 to 30 kPa in the loose placed backfill. Within the expected zone of compacted backfill the undrained shear strengths ranged from 11 to 46 kPa. Vane shear strengths in the organic backfill range from 12 to 39 kPa. A histogram of these shear strengths is shown on Figure 4-12.

4.8.3 Concrete Cores

Thirty concrete cores were recovered from the aqueduct between Mile 85.6 and Mile 95.7. All cores were transported back to UMA Engineering for photographing, measurement of dimensions and unit weight testing. Nineteen cores were subsequently shipped to the University of Alberta for additional testing that included depth of carbonation, petrographic analysis and strength testing (split cylinder tensile strength, compressive strength and shear testing). The test results directly related to this thesis project are discussed below.

The unit weight testing was performed according to ASTM Standard C642-97. The unit weights range from 22.36 to 23.99 kN/m^3 with a mean of 23.35 kN/m^3 and a standard deviation of 0.39 kN/m^3. A histogram of the concrete unit weights is shown on Figure 4-13. The unit weights measured represent the concrete used in the first three years of construction (1915 to 1917). During this period of time approximately 80 percent of the aqueduct was constructed. The statistics for the unit weights indicate that a high degree of uniformity was achieved, which was the objective of the GWWD (Section 3.2.4).

Measurement of the concrete core thickness was done to help determine the variability in the cross-sectional area of concrete. Of the thirty concrete cores recovered from the aqueduct, only eighteen were full depth cores that could be measured to determine the thickness of the concrete in the aqueduct. The measured thicknesses are compared with the original design thicknesses in Table 4-2. Only for four concrete cores were thinner than the design value, but within 25 millimetres of the design value. In general, the concrete cores were thicker than the design section by 30 millimetres, on average. In contrast, the invert core (R213-b) taken at Mile 87.295 was 327 millimetres thicker than design. This was attributed to an error on the record drawing (GWWD, 1919) which showed that a reinforced invert (244 millimetres thick) was constructed at the location where the core was taken. The measured core thickness (571 millimetres) corresponds more closely to a gravity invert (up to 635 millimetres thick) as shown on Figure 3-15. This difference may be explained by the following discussion.

The soil profile between Mile 87 and Mile 88 on Figure 4-1 shows that at a few locations the depth of organics (peat) extended below the foundation grade by about 150 to 450 millimetres. When organic soils were encountered in the original excavation they were completely removed. The excavation could have been backfilled with sand and gravel to the required foundation level. However, because sand and gravel is permeable and the groundwater levels in this area are above the aqueduct and the backfill is organic (low unit weight) the engineers would have known that the conditions required for buoyancy existed (Section 4.2). A note on the gravity invert drawing indicates that in situations like these the thickness of the invert could be reduced if the foundation soil was not highly permeable. This would explain why the concrete thickness was less than the design value of 635 millimetres. Core No. R213-b was not included in the calculation of any statistics because the thickness of the gravity type inverts varied, based on local soil conditions.

4.8.4 Internal Dimensions

The internal dimensions of the aqueduct were measured approximately every 160 metres using specially built telescoping measuring rods. Figure 4-14 shows the specific positions within the aqueduct cross-section where measurements were taken. A summary of the results for the area between Mile 85.0 and the Intake at Mile 97.5 are presented in Table 4-3. Within this area only three aqueduct section types were constructed, the B, R and S sections (Table 3-1). The measurements presented in Table 4-3 are not impacted by the type of invert constructed. On average the internal size of the aqueduct is slightly larger than design. Except for the inside width of the S section, which has a standard deviation of 24.9 millimetres, the standard deviation was less than 20 millimetres. The coefficient of variation, which measures the dispersion of a random variable, was typically less than about 0.5 percent indicating very little dispersion of the data.

The measurements undoubtedly have some error in them. The measurements are considered as indicators as to how the size of the aqueduct may vary from the design values. In general, the internal dimensions indicate that the aqueduct is slightly larger than design. If the concrete thickness is assumed to be as per design then the change in self-weight and buoyant force due to the larger overall section size is about 1.4 kN/m and 3.0 kN/m respectively. That is, the buoyant force increases slightly more than the self-weight. The influence of section size will be further discussed in Chapter 5.

4.8.5 Survey Results

The results of the survey to determine profiles of the backfill berm and the aqueduct invert are shown on Figure 4-2. Figure 4-15 shows the crown backfill depths calculated from the survey data. The results of the groundwater level surveys confirmed that the aqueduct and the majority of the backfill are submerged. Figure 4-16 shows a profile of the groundwater elevations in the Boggy River Stretch that were delineated from survey data of groundwater levels.

4.9 SUMMARY AND DISCUSSION OF RESULTS

The work presented in this chapter was used directly in the buoyancy analysis presented in Chapter 5. The need to review the buoyancy potential of the aqueduct between Mile 85.0 and the Intake at Mile 97.5 was demonstrated in Section 4.3. The components of the buoyancy model used in Chapter 5 were discussed in Section 4.4. The field program undertaken to collect the necessary data to complete the buoyancy analysis was described in Section 4.5. The major difficulties encountered due to the variability in the measured parameters and as a result of the preferred repair strategy (granular ballasting) are discussed below.

The unit weights and moisture contents of the backfill soils measured between Mile 85.0 and the Intake at Mile 97.5 had a very wide range of values (Section 4.8.2). The data could not be used collectively in the buoyancy analysis. The variability in unit weight and moisture content was reduced by grouping the data. The data was grouped to match the extents of the three stretches mentioned in Section 4.6; the Boggy River Stretch, the Summit Stretch and the Snake Lake Stretch. The data groups were subsequently used in the buoyancy analysis for the respective stretches.

The depth of the crown backfill was also highly variable as shown on Figures 4-2 and 4-15. This variability was resolved in the same manner as that for the backfill unit weights. That is, grouping of the data collected from each of the three stretches identified in Section 4.6.

Engineering challenges were also faced due to the range in values of the backfill shear strength (Figure 4-12) and especially the depth of compacted backfill. The undrained shear strengths of cohesive backfill ranged from 11 to 46 kPa indicating that the level of compaction was highly variable. The depth of the compacted backfill zone was difficult to determine with accuracy. Both of these parameters can have a big impact on the buoyancy analysis. A conservative approach was taken due to the variability and difficulty in measuring these parameters. Where cohesive backfill was found, a low undrained shear strength of 12 kPa was chosen. The depth of compacted backfill varies with the type of backfill material that was available from the original excavation. The areas having a high percentage of either organic (for example, the area west of Mile 88.0 on Figure 4-1), west of Mile 88.0 or mineral backfill soils (for example, the area between Mile 92.0 and 94.0 on Figure 4-1) were not a problem. This was not a problem because it was assumed that organic backfill soils would not contribute any shear resistance and in areas where the backfill was mainly mineral in nature, the weight of the backfill was sufficient to prevent buoyancy. The areas of largest concern were those having roughly equal proportions of organic and mineral backfill. For example, the areas between Mile 88.0 and Mile 89.0, and Mile 94.0 to Mile 95.0. Hand auger holes were drilled beside the aqueduct to determine if the mineral soils were preferentially placed at the bottom of the aqueduct trench. The results indicated that this was generally the case. The depth of compacted backfill used in the buoyancy analyses ranged from 0.6 to 1.0 metres, based on the hand auger test hole results and the soil profile shown on Figure 4-1.

The measured values of the unit weight of concrete were quite uniform as indicated by a coefficient of variation = 1.67 percent. As will be shown in Chapter 5, the effect of measuring the concrete unit weights had a significant positive impact on the buoyancy analysis.

The variability in the concrete area and total aqueduct section size could not be measured directly. The concrete thickness and internal dimensions were measured and used as an indicator of the variability in concrete area and total section size. The results of a sensitivity analysis completed to determine the potential variability in these two parameters are presented in Section 5.1.3.

Some of the biggest engineering challenges encountered in the Buoyancy Assessment Program are directly related to the preferred remedial option, granular ballasting. The additional load due to granular ballast results in increased structural stresses, particularly in the invert and the potential for differential settlement. Based on the results of the trial ballasting project (described in the next section) bearing capacity and uniform settlement were not considered to be significant problems. The two potential problems were compounded where the aqueduct was built with non-reinforced inverts, has signs of structural distress (cracked sections) and where the aqueduct is founded on piles or very soft soils. These problems were largely solved by using different repair strategies as described in the next section.

4.10 REMEDIAL OPTIONS – GENERAL OVERVIEW

Three remedial options were identified in advance as possible ways of reducing the risk of buoyancy during extended shutdowns. The three options included drainage, soil anchors and additional weight (ballasting).

Upon first thought, drainage of the surrounding terrain appears to be the most logical approach to addressing buoyancy concerns. However, the topography and groundwater conditions along the majority of the aqueduct alignment between Mile 85.0 and the Intake at Mile 97.5 do not favour drainage. The only locations where drainage could be effective is west of Mile 85.6 and for a short distance on either side of Mile 88. Drainage was not considered as a general solution due to its questionable effectiveness and the requirement for on-going maintenance.

The use of soil and/or rock anchors was considered because they would apply little additional load to the aqueduct except when it became buoyant during extended shutdowns. A soil anchor-testing program was undertaken to determine the potential design load capacities of helical screw type soil anchors. Single helix and multiple-helix anchors having two or three helices spaced at 1.2 metres intervals were tested (Figure 4-17). The diameter of the anchor helices was 400 millimetres. The single helix anchors provided approximately the same capacity as the multiple-helix anchors. The design or allowable capacity (41 kN) for these anchors was approximately one-half of the expected design capacity (80 kN). The cost of the soil anchor option was about four times the cost of granular ballasting. In addition, concerns about transferring the anchor load to the aqueduct and creep of the anchors made this option less attractive.

The third option was to provide additional weight to the aqueduct structure. This could be accomplished by placing granular ballast over the original backfill or by static water ballasting. Static water ballasting consists of damming water inside the aqueduct behind an inflatable dam.

Concerns identified with applying additional load to the aqueduct in the form of granular ballast included the following ultimate (ULS) and serviceability (SLS) limit states; (1) structural capacity (ULS) (2) bearing capacity of the foundation soils (ULS) (3) cracking of the concrete (SLS) and (4) consolidation settlement (SLS). The structural capacity and bearing capacity were reviewed prior to undertaking a trial ballasting project to evaluate the feasibility of placing granular ballast on the aqueduct. The trial ballasting program consisted of ballasting a 200 metre long section of the aqueduct with a 500 millimetre thick layer of granular ballast (Figures 4-18 and 4-19). The granular ballast is similar in size and gradation to railway ballast. The material used had a uniformity coefficient (D_{60}/D_{10}) of 1.67 and a D_{50} of 28 millimetres. The trial section was equipped with vibrating wire piezometers to measure pore water response, mechanically anchored monuments and settlement points (Borros anchors) to measure settlement of the aqueduct and settlement plates to measure compression of the backfill. The monitoring results and an internal inspection of the aqueduct did not reveal any adverse effects that would preclude the use of granular ballast as a feasible option.

Static water ballasting was not thought to be practical for ballasting long stretches of the aqueduct because the pipe could still not be dewatered for more than 48 hours to facilitate aqueduct maintenance repairs. In fact, in many ways it could defeat the very purpose of the dewatering process. Hydraulic capacity of the pipe downstream of about Mile 89 would prevent installation of any permanent weir or dam structures inside the pipe necessary to hold back water. However, upstream of Mile 89 the aqueduct was designed with additional capacity (larger internal cross-section) because the design engineers anticipated the need for a second aqueduct that would be built from Mile 87.45 to Deacon Reservoir. The distinct advantage of static water ballasting over granular ballasting is that no new load is added to the pipe. This avoids increased stress levels within the aqueduct invert that unavoidably accompany granular ballasting. As discussed in Section 4.3, there was a concern regarding invert strength, particularly upstream of Mile 93.5.

The final design of the remediation works involved a combination of drainage work, granular ballasting and static water ballasting to protect the pipe from buoyancy. Granular ballasting was completed over much of the length of the aqueduct determined to be at risk to buoyancy (Figure 4-20). However, granular ballasting was not feasible at all locations. About 800 metres of drainage work was required west of Mile 85.7 due to the condition of the aqueduct at a bend in the pipe alignment and because a portion of the pipe was founded on piles. Concern over the stability of the cracked pipe forming the bend under additional backfill load and the potential for differential settlement and cracking of the pipe on either side of the pile foundation precluded the use of granular ballast.

The issue of invert strength and hydrostatic uplift pressures was discussed in Section 4.3. Because of concerns over invert strength, static water ballasting was used upstream of Mile 93.7. The length of aqueduct located upstream of Mile 93.7 was largely constructed with non-reinforced inverts such as, the 'E' type invert shown on Figure 3-14. This style of invert is susceptible to cracking under hydrostatic heads greater than 3.6 metres. Use of granular ballast in this area would greatly increase the load transferred to the invert through the arch and result in overstressing the inverts. The static water ballast profile required to prevent buoyancy was provided by installing an inflatable rubber dam at Mile 93.7. The dam is operated only when the aqueduct is shutdown. At all other times the dam lies flat on the invert and causes no significant head loss. The aqueduct upstream of the dam can not be dewatered for more than 48 hours at a time. Downstream of the dam the aqueduct can be dewatered as required.

The use of granular ballast as the preferred remedial option to minimize the risk against buoyancy will form the basis of the buoyancy analysis discussed in more detail in Chapter 5.

5.0 AQUEDUCT BUOYANCY ANALYSIS

The method used for buoyancy analysis of the aqueduct evolved out of necessity. Due to the number of variables affecting aqueduct buoyancy, a limit state design (LSD) approach using partial safety factors (PSF) was adopted over a working stress design (WSD) approach that only uses a single global factor of safety. The objective was to provide an economic means of achieving a consistent level of safety (Gore and Storrie 1995). The PSFs used in the original analysis were taken directly from the Ontario Highway Bridge Design Code (OHBDC) (1991), with the exception of the PSF for shear resistance, which was based on engineering judgement. The loading from the required depth of granular ballast calculated using PSFs from the OHBDC (1991) raised concerns over the structural capacity of the aqueduct invert. A review of the aqueduct invert strength and the variability of the parameters in the buoyancy model was completed and a new approach taken to ensure a more consistent level of safety. This new course of action involved determining which parameters in the buoyancy model were most influential and calculating project-specific partial safety factors for these variables based on measured values. The results of this buoyancy analysis were checked using a Monte Carlo simulation.

As discussed in Chapter 4, the Buoyancy Assessment Program for the Shoal Lake Aqueduct included the entire area from Mile 85.0 to the Intake (Mile 97.5). The length of the project was sub-divided into three stretches (Boggy River Stretch, Summit Stretch and Snake Lake Stretch), each having relatively consistent soil and groundwater conditions. The following discussion is based on the work undertaken for the Boggy River Stretch (Mile 85.0 to Mile 88.5). Procedures described in this chapter were also used for the buoyancy analysis work completed for the Summit and Snake Lake stretches. Some minor changes to the analysis were required to reflect the different soil and groundwater conditions in these stretches.

For the purpose of this thesis, four different analyses were completed to show how the level of safety between methods can vary. Alternative LSD methods were used for three of the analyses. The WSD method was used for the fourth analysis.

5.1 PRELIMINARY ANALYSIS

Several stages were identified in Chapter 2 that can be used to aid the application of LSD procedures to engineering problems where no design codes exist. Such is the case for assessing the buoyancy potential of the aqueduct. These stages include:

1. Identifying all possible limit states, including ultimate and serviceability limit states.
2. Selection or development of a model(s) to evaluate each limit state.
3. Identifying all possible variables or uncertainties.
4. Sensitivity analyses to determine which variables have the greatest influence on the model output.
5. Data collection and interpretation based on the results of the sensitivity analyses.
6. Selection of a target level of safety.
7. Analysis and design.

Stages 1 to 5 are discussed in Section 5.1. Stages 6 and 7 are incorporated into the later sections of this chapter.

5.1.1 Applicable Limit States

The ultimate limit state (ULS) that is the focus of this thesis is buoyancy of the aqueduct. Other ULSs are associated with the remedial repairs (granular ballasting) used to reduce the risk of buoyancy. The ULSs associated with granular ballasting include foundation bearing capacity and structural capacity of the aqueduct. The serviceability limit states (SLS) include settlement and cracking of the aqueduct. The bearing capacity and settlement limit states were evaluated using traditional geotechnical methods and a full scale trial ballasting program. The trial ballasting program consisted of placing 500 millimetres of granular ballast over the aqueduct. No significant settlements, typically less than 5 to 10 millimetres, were observed in the trial ballasting stretch over a monitoring period of 1.5 years. The structural capacity ULS and cracking SLS required a finite element structural analysis. This SLS was partially evaluated during the trial ballasting program, which did not result in any new cracks on the inside of the structure. The finite element analysis was completed by CH2M_Gore and Storrie Ltd. of Toronto, Ontario. The consulting firm CH2M_Gore and Storrie Ltd. are partners in the Shoal Lake Aqueduct Condition Assessment and Rehabilitation Program along with UMA Engineering Ltd. and the City of Winnipeg – Water and Waste Department.

5.1.2 General Buoyancy Equation and Variables Affecting Analysis

The general buoyancy concept for the aqueduct was discussed in Chapter 4 where four resisting forces and one disturbing force were identified. The resisting forces include: backfill weight, backfill shear resistance, aqueduct weight and the weight of any granular ballast required to achieve the target level of safety against flotation. The disturbing force is the buoyant force acting on the aqueduct.

The general buoyancy equation used in the buoyancy analysis is:

[5-1] (backfill weight + backfill shear resistance + aqueduct weight + granular ballast weight) $\geq$ buoyant force

All terms have units of (kN/m). The backfill weight term includes two components, the crown backfill and the arch leg backfill (Figure 4-4). The crown backfill weight is calculated as follows:

[5-2] Crown backfill weight (kN/m) = $\gamma_{backfill} W_{external} D_{backfill}$

and the arch leg backfill weight is calculated as follows:

[5-3] Arch leg backfill weight (kN/m) = $\gamma_{backfill}$ ($W_{external}$ $H_{external}$ + A_{haunch} – $A_{aqueduct}$)

where:

$\gamma_{backfill}$ (kN/m^3) - bulk or submerged backfill unit weight depending on degree of submergence

$W_{external}$ (m) - largest horizontal dimension of the aqueduct (Figure 3-7)

$D_{backfill}$ (m) - depth of backfill above the aqueduct crown

$H_{external}$ (m) - vertical dimension from the outside edge of the invert to the aqueduct crown (Figure 3-7)

A_{haunch} (m^2) - excavation area for the invert below the base of the arch legs (Figure 3-7)

$A_{aqueduct}$ (m^2) - total cross-section area of the aqueduct

For the purpose of calculating the backfill weight component, the aqueduct dimensions and areas were assumed to be equal to the design values. This is a reasonable assumption because any changes in backfill area due to changes in the aqueduct dimensions are small relative to the total backfill area. The backfill zone assumed to contribute to the resistance is that contained within the area defined by vertical projections from the outside corners of the aqueduct invert (Figure 4-4). The uncertainties related to backfill weight are the backfill unit weight and the crown backfill depth.

The backfill shear resistance term is calculated as follows:

[5-4] Backfill shear resistance (kN/m) = (τ $D_{compacted}$) 2

Where:

τ (kPa) - shear strength of compacted mineral soil layer

$D_{compacted}$ (m) - depth of compacted backfill

The calculated backfill shear resistance is doubled to account for the shear resistance on both sides of the aqueduct. The shear strength was based on measured values but assumed to be zero where the backfill soils are organic in nature. The depth of compacted backfill was assumed to be 1.0 metres, that is, about 80 percent of the compacted depth (1.2 metres) that was required in the original construction specifications for the aqueduct (GWWD 1914). The assumptions regarding the shear failure surface were discussed in Chapter 4. The shear resistance from the granular ballast is small (less than 2 kN/m) because of the low normal stresses acting on the assumed vertical shear plane. As a conservative solution, no component of shear resistance was included from the granular ballast.

The aqueduct weight is calculated as follows:

[5-5] $$\text{Aqueduct weight (kN/m)} = \gamma_{concrete} A_{concrete}$$

Where:

$\gamma_{concrete}$ (kN/m^3) - saturated unit weight of concrete

$A_{concrete}$ (m^2) - cross-sectional area of concrete

A saturated unit weight for concrete was used because the aqueduct is completely submerged within the area under consideration (Mile 85.0 to Mile 97.5). Both of the parameters in Equation 5-5 are considered as uncertainties in the buoyancy model. The variability of the cross-sectional area has been handled differently in the buoyancy models discussed below.

The granular ballast weight is calculated as follows:

[5-6] $$\text{Granular ballast weight (kN/m)} = \gamma_{ballast} W_{external} D_{ballast}$$

where:

$\gamma_{ballast}$ (kN/m^3) - bulk or submerged ballast unit weight depending on degree of submergence

$W_{external}$ (m) - largest horizontal dimension of the aqueduct

$D_{ballast}$ (m) - depth of ballast

The width of ballast contributing to the resistance is assumed to be equal to the width of the aqueduct at its widest point, the same width used in calculating the backfill weight term. The depth of ballast is equal to or greater than that required to ensure the resisting force is equal to or greater than the buoyant force. The main uncertainty is with the unit weight of ballast. The depth of ballast is a calculated quantity and if necessary can be adjusted during construction to ensure the required ballast weight is applied to the aqueduct.

The buoyant force is calculated as follows:

[5-7] Buoyant force (kN/m) = $\gamma_{water} A_{aqueduct}$

Where:

γ_{water} (kN/m^3) - unit weight of water, 9.81 kN/m^3

$A_{aqueduct}$ (m^2) - total cross-sectional area of the aqueduct

The only uncertainty in calculating the buoyant force is the total cross-sectional area of the aqueduct because the unit weight of fresh water is a well known quantity. The variability of the total cross-sectional area was handled differently in the buoyancy models discussed below.

A resisting ratio is used to evaluate the buoyancy ULS as shown in Equation 5-8. Resisting ratios greater than 1.0 indicate that the resisting forces are greater than the buoyant force. Values less than 1.0 indicate that granular ballast is required to provide a minimum resisting ratio of 1.0.

[5-8] $$\text{Resisting Ratio, RR} = \frac{\text{Total factored resisting force (kN/m)}}{\text{Factored buoyant force (kN/m)}}$$

In summary, the variables or uncertainties in the general buoyancy equation include: backfill unit weight, crown backfill depth, compacted backfill shear strength, compacted backfill depth, concrete unit weight, concrete area, granular ballast unit weight, and the total cross-sectional area of the aqueduct. One additional uncertainty not evident in the above equations is the degree of submergence of the backfill and the aqueduct. This is discussed in Section 5.2.1.

With the possible exception of soil shear strength, the measurement uncertainties associated with these variables are considered to be reasonably small because unit weights and dimensions can be measured directly using simple procedures. Uncertainty in the buoyancy model is assumed to be small because the problem essentially involves weight calculations and buoyant force calculations. Aside from any variation in the total cross-sectional area of the aqueduct, the buoyant force can be calculated with accuracy using Archimedes principle. Uncertainties associated with measurements or the model are considered to be small and have not been accounted for in the buoyancy analysis.

5.1.3 Sensitivity Analysis To Determine Parameter Significance

In order to assess which of the uncertainties has the most impact on the buoyancy analysis, and which ones are worth while measuring, a Monte Carlo simulation was completed using @ Risk Version 4.5 (Palisade Corporation 2002). This software works in conjunction with spreadsheet programs and allows probability distributions to be specified for each uncertainty or parameter in any equation. Each time an equation is calculated, that is for each simulation cycle, a random value from each probability distribution is used. Correlation coefficients between different variables can also be specified. A more complete discussion regarding the set up of a simulation model can be found in Section 5.2.5. This sensitivity analysis is significantly different than the one originally developed. It represents the use of actual data that would have been available at this point in the Buoyancy Assessment Program. It is considered to be original work completed by the author for use in this thesis.

The sensitivity analysis was completed for one aqueduct section type, the 'BO' aqueduct section shown on Figure 3-10, to determine the significance of each parameter in Equation 5-1. It is reasonable to assume that the results will be applicable to all the aqueduct section types that were constructed between Mile 85.0 and the Intake (Mile 97.5). Normal probability distributions were assigned to the following input parameters: saturated unit weight of backfill, depth of crown backfill, shear strength and unit weight of concrete. Triangular distributions were assigned to the area of concrete and the total cross-section area of the aqueduct.

The probability distributions and statistics for the input parameters are summarized in Table 5-1. These distributions are based on preliminary information that was available at the beginning of the project and assumed values for the mean and the coefficient of variation. The preliminary information included:

- measured backfill unit weights (saturated);
- measured backfill unit weights (saturated);
- shear strengths from the detailed test holes (UMA 2000);
- crown backfill and compacted backfill depths based on archived information (GWWD, 1914 and 1919) and observations made during site reconnaissance trips;
- concrete unit weights based on measured densities from concrete cores previously collected from the aqueduct between Mile 24 and Mile 70 (Gore and Storrie, 1995).

These uncertainties were defined with normal probability distributions truncated to reflect realistic maximum and minimum values. The term for granular ballast weight (Equation 5-6) was excluded from the analysis because the objective was to determine which uncertainties have the most influence in determining which areas of the aqueduct are at greatest risk to buoyancy.

The variability of the concrete area and the total aqueduct section area was estimated from the results of a separate sensitivity analysis. This analysis was based on an assumption that the concrete thickness and internal dimensions are within +/- 50 millimetres of the design values. Nine possible combinations of concrete thickness and internal dimensions were evaluated. A cross-section of the aqueduct was drawn for each case and the resulting concrete and total section areas were measured using AutoCad Version 14. The results are presented in Table 5-2. The results of the analysis show that the concrete and total section areas have a correlation coefficient of 0.77. The maximum and minimum values for the concrete and total section areas were used, along with the mean design values to specify the parameters for a triangular probability distribution.

A correlation coefficient of 0.77 was used to relate the concrete area and the total aqueduct section area input variables. A correlation coefficient of 0.9 was used to relate saturated backfill unit weight, backfill shear strength and compacted backfill depth. This was done to reduce the possibility of sampling a high backfill shear strength when a low backfill unit weight is sampled. In this way, the results would conform with the assumption that aqueduct sections backfilled with organic soil would have no shear resistance (Section 4.4).

The simulation model was set up in a manner where Equations 5-2 to 5-8 (excluding Equation 5-6) were calculated separately. For each simulation cycle, the probability distribution for each variable or input parameter was sampled and these values used to calculate each of the equations in the model. Fixed values were defined for the maximum width of the aqueduct and the area of the arch leg backfill.

Two simulations, each with 30,000 simulation cycles were completed. The first simulation included the backfill shear resistance, which was not included in the second simulation. The backfill shear resistance was excluded in the second simulation because the majority of the Boggy River Stretch has organic backfill that was assumed to have no shear strength. The output results available from the @Risk Version 4.5 (Palisade 2002) software package include: histograms of the sampled values for each input parameter, summary statistics and histograms for each of the calculated values, detailed input and output statistics and results from multi-variate stepwise regression and rank order correlation sensitivity analyses.

For the purpose of this parameter sensitivity analysis, the most important results are those from the multi-variate stepwise regression and rank order correlation analyses. These results are summarized in Table 5-3. These results for the resisting ratio should be the best indicator as to which parameters are most important to the buoyancy analysis. This is because the resisting ratio is calculated using the resisting forces and the buoyant force.

The values of the regression coefficient can range from –1 to +1. Values equal to zero or near zero indicate that the input variable has little to no relationship with the output variable. That is, a change in the value of the input variable has little impact on the resisting ratio. Values closer to –1 or +1 have significant impact. A value of 1 would indicate that the output would change by 1 standard deviation for a 1 standard deviation change of the input parameter. High values of the square of the correlation coefficient (R^2) determined from regression analysis indicate a strong linear relationship between the input and output variables.

Correlation coefficients can also range from –1 to +1. A value of zero would indicate no correlation between the input and output variables. Values of –1 or +1 indicate respectively completely negative or positive correlations. All other values indicate partial correlations that become more significant as the value approaches 1 or –1.

The results from the simulations are summarized in Table 5-3. The square of the correlation coefficient is close to 1 indicating a good relationship between the inputs and the resisting ratio. The regression coefficients for the simulation that included shear resistance indicate that the two most influential parameters are the saturated backfill unit weight and the shear strength. These are followed in influence by the crown backfill depth and the compacted backfill depth. The concrete area, concrete unit weight and total section area appear to have the least influence. The regression coefficients from the second simulation (no shear) indicate that the saturated backfill unit weight and crown backfill depth are the dominant parameters. Surprisingly, the concrete unit weight and total section area are not as significant. This is discussed below in more detail. Due to the correlation coefficients used to relate the some of the input parameters, the correlation coefficients summarized in Table 5-3 are not viewed as being good indicators of parameter significance. This is most obvious in the results for the simulation that included shear resistance, the three backfill parameters that were correlated together also have highest correlation coefficients relating them to the resisting ratio.

The statistics for the input and output variables are presented in Table 5-4. The first observation is that the four most influential input parameters also have the highest coefficients of variation. This confirms the results of the regression analysis (Table 5-3) based on the meaning of the value of a regression coefficient. For example, if the concrete unit weight changed in value by 1 standard deviation it would have less impact on the resisting ratio than if the saturated unit weight of the backfill changed by 1 standard deviation. This is valuable information, but when looking at the statistics in Table 5-4 for the output (resisting forces) it is obvious that the weight of the aqueduct can account for anywhere from 20 to 90 percent of the total resisting force. This clearly becomes more important as the unit weight of the saturated backfill decreases.

Conclusions drawn from the sensitivity analysis include:

1. Reducing or confirming the dispersion (uncertainty) in the saturated backfill unit weight, shear strength, crown backfill depth and the compacted backfill depth will produce more reliable buoyancy analysis results that can be used to identify areas of the aqueduct requiring granular ballasting repairs.
2. Collection of additional data should help to confirm the dispersion of the most influential input parameters (saturated backfill unit weight, shear strength, crown backfill depth and compacted backfill depth)
3. Collection of field data to confirm the aqueduct size and concrete unit weight is important because the size and weight of the aqueduct are significant components of the total resisting force. The aqueduct size also influences the buoyant force.

5.1.4 Data Collection and Interpretation

The main purpose of presenting the sensitivity analysis for parameter significance at this point in the thesis is to show a logical pattern of events that should take place in developing the buoyancy model. In actual fact, during this stage of the Buoyancy Assessment Program, much of the following information had already been collected and used in the original buoyancy analysis that is discussed in Section 5.2.3. The work described in Section 5.2.4 explains why the sensitivity analysis was undertaken and what additional information was collected. The text below is written from the perspective that minimal information had been collected beforehand. However, it is representative of the actual work completed over the course of the Buoyancy Assessment Program.

The field program undertaken to collect the additional data mentioned in the conclusions at the end of the previous section has already been presented in Chapter 4. The field work was completed along the entire stretch of aqueduct from Mile 85.0 to the Intake at Mile 97.5. For convenience, the following sentences provide a brief summary of the field program. Samples of the crown backfill were taken to measure their saturated unit weights (Figure 4-10). The dry unit weights of the measured samples were used in a regression analysis to derive a relationship between dry unit weight and water content of the backfill samples (Section 4.8.2 and Figure 4-11). This relationship was used to predict dry unit weights and hence saturated unit weights based on water contents from other backfill samples. The unit weights of the crown backfill varied considerably along the entire stretch of the aqueduct from Mile 85.0 to the Intake at Mile 97.5. Grouping the backfill unit weights from each stretch, such as the Boggy River Stretch, reduced the dispersion of the backfill unit weights within each stretch. Undrained shear strengths of the backfill soils were measured in areas where mineral backfill soils were encountered (Figure 4-12). The depth of compacted backfill was difficult to delineate, as described in Section 4.9. The areas where shear resistance became important was in areas where there was a mixture of organic and mineral soils. In these areas, the depth was first estimated from the soil profile on Figure 4-1 and then checked by drilling hand auger holes beside the aqueduct. Where the backfill was predominantly mineral in nature the weight of the backfill was sufficient to prevent buoyancy. Where the backfill was mainly comprised of organics a shear resistance of zero was used. The crown backfill depths were delineated by completing a profile survey of the backfill berm along centerline of the aqueduct (Figures 4-2 and 4-15). The unit weight (Figure 4-13) and thickness (Table 4-2) were measured for each concrete core taken from the pipe. Internal measurements of the aqueduct previously collected for hydraulic modeling of the pipe, were used to aid in determining the variability in the pipe size (Table 4-3). The concrete thickness and internal measurements were not used in the original work completed at UMA Engineering Ltd. but have been incorporated into subsequent work undertaken for this thesis.

This information has been used in developing the various buoyancy models based on LSD that are discussed in the next section. A more complete discussion on how the data were used with each of the models is presented in following sections.

5.2 LIMIT STATES DESIGN

Three LSD methods have been completed to show how the uncertainties in the variables can be accounted for. As mentioned previously, for the purpose of demonstrating these methods, only one area of the aqueduct between Mile 85.0 and the Intake (Mile 97.5) was analyzed. This area is the Boggy River Stretch (Mile 85.0 to Mile 88.5). The principles for the methods shown below can be applied to the Summit and Snake Lake stretches with only minor modifications to account for the different site conditions.

The first two of the three LSD methods are based on the partial safety factor method presented in Section 2.4.4. The two methods are essentially the same as they use partial safety factors (PSFs) to account for uncertainties in the various loads and resistances. The first method utilizes the PSFs provided in the OHBDC (1991) and the second method utilizes PSFs calculated specifically for this project. The first method is presented for two reasons. One, the original buoyancy analysis completed for the aqueduct used the PSFs from the OHBDC (1991). Two, it shows that the degree of uncertainty in the results of the analysis can be higher when using PSFs developed for different conditions. The second method uses project-specific PSFs that take into account the variability of the uncertainties in the buoyancy model.

The third LSD method used is a Monte Carlo simulation similar to a level 2 or level 3 reliability-based design (Section 2.4.4). This method was completed to verify the results obtained using the project-specific PSFs developed for the buoyancy analysis and to confirm the target level of safety.

5.2.1 General Considerations for the Boggy River Stretch (Mile 85.0 to Mile 88.5)

Some general considerations for the Boggy River Stretch had to be taken into account for the buoyancy analysis. The aqueduct in this area consists of two aqueduct section sizes, the R and the B sections (Section 3.2.2). The various invert sections used with the R and B aqueduct sections include: RPM, RRM, RA, RE, RG, BO, BP, BPM and Bpiled (Figures 3-10 to 3-15 and 3-27; Table 3-1). Each of these sections had to be evaluated separately due to the different dimensions, concrete areas and total section areas.

This stretch of the aqueduct has predominantly organic backfill, whereas the other stretches have considerably more mineral backfill. To reduce the dispersion of the backfill unit weights, only unit weights measured within this stretch have been used. As mentioned previously, it has been assumed that shear resistance of organic backfill could not be relied upon as a resisting force. Therefore the majority of the Boggy River stretch was assumed to have zero shear resistance. However, east of Mile 88.0 the soil stratigraphy above the aqueduct invert level (Figure 4-1) shows an increase in the amount of mineral soils that could have been used as compacted backfill, particularly east of Mile 88.26. This was verified by drilling hand auger test holes beside the aqueduct. A small amount of shear resistance was therefore included in the buoyancy model from about Mile 88.26 to Mile 88.5.

As discussed in Section 4.6, the Boggy River Stretch is generally poorly drained (Figures 4-7 and 4-16) except for two areas. The first area is located between Mile 85.0 and Mile 85.5. The second area is between Mile 87.95 and 88.05. Within these two areas the groundwater levels are just at the top of the aqueduct. At all other locations the groundwater levels are near the top of the crown backfill berm. For the buoyancy analysis, the aqueduct and the arch leg backfill can be considered as fully submerged at all locations. Submerged unit weights for the backfill have been used for all areas except within the two areas defined previously, where the groundwater levels are just at the top of the pipe. Within these two areas, bulk unit weights for the crown backfill were used. The groundwater levels used in the buoyancy analysis were based on measured values and a hydrology study (UNIES, 1998). Some improvements to the drainage ditches west of Mile 85.5 were required to achieve the groundwater levels used in the analysis. These improvements were required for other reasons that are described in the next paragraph.

Some structural aspects of the aqueduct had to be considered as well. From Mile 85.48 to Mile 85.61, approximately 214 metres of the aqueduct was constructed with a timber pile foundation (Figure 3-27) and from Mile 85.63 to Mile 85.66 (length = 70 metres) there is a bend in the aqueduct alignment. The four aqueduct segments forming the bend are cracked. Due to the potential for differential settlement of the aqueduct on either side of the piled section and the concerns regarding structural stability of the cracked segments in the bend, the area west of Mile 85.7 was not considered for granular ballasting. As discussed previously, some drainage improvements were completed to reduce the groundwater levels to the top of the aqueduct in this area. The increased weight of the crown backfill resulting from lowered groundwater levels was sufficiently significant to ensure that the total resistance was greater than the buoyant force.

The concrete area and total cross-sectional area of the aqueduct were difficult to determine due to the curved shape of the pipe and because the pipe is buried. However, as discussed in Chapter 4, the thickness of the concrete cores taken from the aqueduct were generally larger than the design thickness. Only four of the cores were thinner, but still within 25 millimeters of the design thickness (Table 4-2). The internal measurements of the aqueduct summarized in Table 4-3 show that the internal size is typically about 12 millimetres larger than design for the R and B aqueduct sections used in the Boggy River Stretch. The concrete area and total section area are needed for the buoyancy analysis. They are somewhat related to the concrete thickness and the internal dimensions. The measured thickness of the concrete cores and internal dimensions indicate that the aqueduct was constructed within reasonable tolerances. Based on this information, a sensitivity analysis was completed to estimate a range of values for the concrete area and total section area. As described in Section 5.1.3 nine different cases were checked, assuming that the concrete thickness and pipe dimensions could not deviate more than +/- 50 millimetres from design. The results are included in Table 5-2. The variability of the concrete area and total section area were handled differently in each method of analysis. They will be discussed separately for each method.

5.2.2 Target Level of Safety

The target level safety chosen for this project is equivalent to a probability of failure of about 0.011 percent or about 1/9090 (1.1×10^{-4}). This target level of safety is equivalent to a reliability index, $\beta = 3.5$ as shown in Table 2-1 (Becker, 1996b). Design codes based on this level of safety and a 50 year design life include the OHBDC (1991) and the CHBDC (2000) (Becker et al., 1998). The term target level of safety is used because the actual probability density functions or distribution curves are not known and therefore the true value of the reliability index, β can not be determined directly (Becker, 1999b).

5.2.3 Partial Safety Factor Method Using OHBDC Load and Resistance Factors

In the early stages of the Buoyancy Assessment Program, it was recognized that there were a large number of variables or uncertainties that should be considered in the buoyancy analysis. A decision was made to use an LSD method employing PSFs to provide a more consistent level of safety than could be achieved using WSD methods. The original work was based on the PSFs provided in the OHBDC (1991). This work has been included to show the impacts of using PSFs developed from different data sets for each variable. In this context, it is interesting to note that Day (1997) felt it would be impossible to specify PSFs that could be applied in general to all sites. In keeping with the chain of events that occurred during the Buoyancy Assessment Program , this analysis has been completed using the original assumptions and design values prior to collecting additional information on concrete unit weight and aqueduct dimensions.

5.2.3.1 Buoyancy model

The buoyancy model for this method was set up using a spreadsheet (Appendix 5-1). As mentioned in Chapter 3, the aqueduct was built in 13.7 metres segments. Each segment has an identification number, for example B0-1142. Sufficient information was available for each aqueduct segment in the Boggy River Stretch to calculate the resisting forces and buoyant force for each segment. The following information was entered for each aqueduct segment: aqueduct mileage, segment number, crown and invert elevations, aqueduct design dimensions, concrete and total section areas based on design dimensions and the elevation for the backfill berm.

The uncertainties in Equations 5-2 to 5-7 were accounted for by applying PSFs to the resistances on the left side and to the load on the right side of the general buoyancy equation as shown below in Equation 5-9.

[5-9] $[\phi_{backfill}$ (backfill weight) + ϕ_{shear} (backfill shear resistance) + $\phi_{concrete}$ (aqueduct weight) + $\phi_{ballast}$ (granular ballast weight)] $\geq \alpha$ (buoyant force)

Where:

ϕ - resistance factor (RF)

α - load factor (LF)

Crown and arch leg backfill weights (Equations 5-2 and 5-3)

All aqueduct dimensions used in the calculations were based on the design values entered for each aqueduct segment. The depth of the crown backfill was determined for each segment by taking an average of the surveyed backfill berm elevations for each segment and subtracting the crown elevation of the pipe. Because a unique backfill depth was available for each segment it was not necessary to use a resistance factor (RF) to account for the crown backfill depth. The backfill unit weights were entered for the corresponding aqueduct segment from which the sample was taken. These cells in the spreadsheet are highlighted. Between segments having measured backfill unit weights, the unit weight was interpolated. To indicate where the aqueduct and the crown backfill berm are completely submerged, the backfill weight values in the spreadsheet are shaded. In areas where only the aqueduct and the arch leg backfill are submerged the backfill weight values are lightly shaded. The minimum prescribed RF of 0.80 OHBDC (1991) for earth fill (dead load weight) was applied to the backfill weight term (Table 5-5).

Backfill shear resistance (Equation 5-4)

As discussed in Section 5.2.1 no backfill shear resistance was included west of about Mile 88.26. East of this point a small amount of shear resistance was included and a conservative undrained shear strength of 12 kPa was used, which was based on measured values (Figure 4-12). The depth of compacted backfill was set at 1.0 metre.

The OHBDC (1991) did not have a prescribed RF for shear resistance therefore a relatively conservative value of 0.5 was selected based on engineering judgement.

Aqueduct weight (Equation 5-5)

The unit weight of concrete was based on the average of the measured unit weights from concrete cores taken from the aqueduct between Mile 24 and Mile 70. The concrete area was assumed to be equal to the aqueduct design value. The minimum prescribed RF of 0.90 OHBDC (1991) for cast-in-place concrete (dead load weight) was applied to the aqueduct weight term (Table 5-5).

Granular ballast weight (Equation 5-6)

The granular ballast weight was only calculated for segments having a resisting ratio less than 1 (Equation 5-8). The unit weight was based on the average of measured values from previous granular ballasting work and the depth of ballast is a calculated quantity. The minimum prescribed RF of 0.80 OHBDC (1991) for earth fill (dead load weight) was applied to the granular ballast weight term (Table 5-5).

Buoyant force (Equation 5-7)

A unit weight of water = 9.81 kN/m^3 was used. The total sectional area of the concrete was assumed to be equal to the design value. A load factor (LF) of 1.05 was applied to account for any uncertainties in total section area. This value was based on the range of LF values (0.90 to 1.10) for hydrostatic pressure in the OHBDC (1991). Values less than 1.0 are not applicable to this work because the buoyant force is a disturbing force. A value of 1.10 was thought to be too severe, therefore an intermediate value of 1.05 was selected (Table 5-5).

The PSFs presented in Table 5-5 were used directly in Equation 5-9 as indicated.

5.2.3.2 Analysis Results

The ULS of buoyancy was checked using the resisting ratio (Equation 5-8). If the resisting ratio is 1 or greater then sufficient weight exists to prevent buoyancy. If the resisting ratio is less than 1, additional weight (granular ballast) is required to prevent buoyancy. A resisting ratio and corresponding granular ballast depth (if required) were calculated for each aqueduct segment so that all segments had a resisting ratio of at least 1. The results are shown graphically on Figures 5-1 and 5-2.

Figure 5-1 shows the resisting ratio profiles with and without granular ballast. The erratic pattern of the profiles is directly related to the variable backfill depths that are illustrated on Figure 4-2. The large spikes in the profile indicate areas having relatively high backfill unit weights. These spikes also correspond to the change in soil profile shown on Figure 4-1. Some of the less pronounced deviations in the profile are a result of the changing aqueduct sections types as illustrated by the aqueduct weight profile in Figure 5-1.

The minimum depth of granular ballast required to provide a resisting ratio of 1 is shown on Figure 5-2. In general, the area from Mile 85.7 to 88.0 requires additional weight. As mentioned previously, the area west of Mile 85.7 was improved by assuming the groundwater levels could be reduced to a level no higher than the aqueduct crown. A small amount of granular ballast was still required near Mile 85.1 due to a shallow depth of existing backfill. Ballast was included east of Mile 88.0 because the existing backfill berm was badly eroded in places and required improvement.

The profile of the design depth of granular ballast shown on Figure 5-2 was established by taking a more global view of the area, recognizing that there is still some uncertainty in the various parameters used in the buoyancy model. An example of uncertainty would be the interpolated backfill unit weights. The objectives in establishing the design profile for the ballast were to ensure the majority of the peaks were covered and to provide a reasonably uniform depth of ballast that would be straightforward to construct. The risk associated with not placing ballast over the short area where the resisting ratio is greater than 1 was deemed to be quite high relative to the cost of placing ballast. In addition, it would be difficult to delineate the exact limits of these areas.

5.2.4 Partial Safety Factor Method Using Project-Specific Partial Safety Factors

The granular ballast depths calculated from the buoyancy analysis using OHBDC (1991) partial safety factors (discussed above) raised concerns about the strength of the aqueduct invert, especially the non-reinforced inverts like the BE invert section shown on Figure 3-14. A review of the uncertainties in the buoyancy analysis was completed and a sensitivity analysis was undertaken, as discussed in Section 5.1.3. The results and conclusions of the sensitivity analysis prompted some additional field work to supplement existing information (backfill unit weights, shear strength) and to collect concrete core samples for the measurement of unit weights and thicknesses. Internal measurements were available and used in this thesis to help estimate the variability of the concrete area and total section area.

In addition, a decision was made to calculate project-specific PSFs that would better represent the variability of the measured parameters in the buoyancy model and also provide a more consistent level of safety. Calculation of the PSFs was very applicable to this project because the structure had already been constructed. This permits direct measurement of many of the parameters in the buoyancy model. The importance of this decision is reflected in the values of the PSFs shown in Table 5-6. The differences are attributed to the dispersion of the measured parameters as indicated by the coefficients of variation, which are also shown in Table 5-6. The PSFs prescribed in the OHBDC (1991) were determined using a coefficient of variation of approximately 0.15.

5.2.4.1 Calculations of Partial Safety Factors

The PSFs were calculated using the second moment probabilistic method described in Section 2.4.4. This method is also known as a level 2 reliability-based design. This method is well known and has been used to develop PSFs for design codes such as the OHBDC (1991). Equations 2-7 and 2-9 were used to calculate the PSFs for resistance

(Ravindra and Galambos, 1978). These equations are repeated below for ease of reference. For this analysis the load factor (LF) was based on engineering judgement and the results of a sensitivity analysis. The equations for calculating LFs can be found in Section 2.4.4.

[2-7] $\phi = \delta e^{-\beta\theta V}$ (log-normal distributions)

[2-9] $\phi = \delta (1-\beta\theta V)$ (normal distributions)

where:

ϕ - resistance factor (RF)

δ - bias factor (mean value / nominal value for each variable)

β - reliability index

θ - separation coefficient (typical range of 0.6 to 0.8)

V - coefficient of variation (standard deviation / mean)

The first step in determining the resistance factors (RFs) was to collect data for each variable to be considered. The data for concrete unit weight, backfill unit weight, and granular ballast unit weight are summarized graphically in Figures 4-13, 5-3 and 5-4. The concrete unit weight and granular ballast unit weight have been assumed to follow a normal distribution while the backfill unit weight more closely resembles a log-normal distribution. Partial safety factors were not calculated for shear strength, concrete area or total section area for reasons discussed below. A bias factor of 1.0 was selected because the RFs are applied to the mean measured value, not a design or nominal value. A reliability index, β of 3.5 was selected as discussed in Section 5.2.2. A separation coefficient, θ of 0.6 was selected based on work by Ravindra and Galambos, 1978. The coefficient of variation, V was based on the statistics from each variable. The calculated RFs are presented in Table 5-6.

Three sensitivity checks were completed to determine the effect of varying either the reliability index, the separation coefficient or the distribution type for each RF that was calculated.

Figure 5-5 shows how each RF would vary for different values of β. The RF design values correspond to β values that are slightly higher than used in calculating the RF.

This results in a slightly increased target level of safety. The curves in Figure 5-5 are fairly flat and show that the calculated RF would not change significantly for values in β ranging from about 3.0 to 4.0.

Figure 5-6 shows how each RF would vary for values of θ ranging from 0.5 to 1.0. The RF is inversely proportional to the value of θ and the slopes of the lines increase with the coefficient of variation. The slopes are greater for the resistance factors calculated using the log-normal equation (Equation 2-7). Using the RFs for backfill unit weight for example, increasing θ from 0.6 to 0.8 would result in a decrease of the RFs by about 0.08. The effect of reducing the RF for backfill from 0.20 to 0.12 only increased the depth of granular ballast required by about 50 millimetres. The impact was small because the majority of the backfill is organic in nature with a submerged unit weight of about 1 to 2 kN/m^3. The separation function has little impact on the RFs for the concrete and granular ballast unit weights that were calculated using Equation 2-9.

Figure 5-7 shows how the RF changes with coefficient of variation for log-normal distributions (Equation 2-7) and normal distributions (Equation 2-9). For coefficients of variation less than about 0.10 both equations return nearly the same value. Therefore, the selection of distribution type for the concrete and granular ballast unit weights was not of much consequence. However, for variables having higher coefficients of variation the selection of the distribution type is important. For example, using Equation 2-9 (normal distribution) to calculate a RF for the submerged backfill unit weight would have resulted in a negative value for the RF. This would have resulted in negative backfill unit weights, a physically impossible situation.

Partial safety factors were not calculated to account for uncertainties in the shear strength or the concrete and total section areas due to the lack of confidence in the data that has been obtained (undrained shear strengths, concrete thickness and internal dimensions). The assumptions regarding shear resistance were not changed from those used in the original buoyancy analysis using the RFs from the OHBDC (1991) (Section 5.2.3). The concrete thickness and internal dimensions indicated that the aqueduct was built to reasonable tolerances. This is not unexpected because of the method of construction used (steel forms). Also, the designing engineers were very cost conscious

(Section 3.2) and the hydraulic design of the aqueduct would require the internal shape and size be within acceptable tolerances.

As discussed in Section 5.1.3, the potential variation in concrete area and total section area was investigated by determining the concrete area and total size for nine different cases (Table 5-2). It was assumed that the overall thickness of the concrete or the internal dimensions did not vary by more than +/- 50 millimetres, a reasonably conservative assumption considering the method of construction (steel forms) and the design thickness of concrete in a typical aqueduct cross-section (Figure 3-10). The results showed that the concrete area, and hence the aqueduct weight, could vary by approximately +/- 20 percent of the design value (Case A). The total section area, and hence the buoyant force, could vary by approximately +/- 3 percent of the design value (Case A). These two parameters are related because any change in concrete thickness results in a change in the total section area. The results indicate a correlation coefficient of 0.77 between these two parameters.

Applying a RF to the concrete area that is less than 1 and a LF to the total section area that is greater than 1 would compound the effects because the two parameters are related. That is, a larger total section area typically corresponds to a larger concrete area. Since the concrete unit weight, and hence the aqueduct weight, is already reduced with a RF of 0.95 it is more reasonable to apply a LF to account for changes in the total section size. This was done by applying a LF of 1.03 to the buoyant force based directly on the results of the 9 scenarios, where the maximum buoyant force was 3 percent higher than the buoyant force based on the design values.

As shown in Table 5-6, two RFs were calculated for the ballast and backfill unit weights, one for bulk unit weights and one for submerged unit weights. This was necessary because there are a few areas in the Boggy River Stretch where the groundwater levels do not rise above the aqueduct crown (Section 5.2.1). In these areas, the RFs for bulk unit weights were used to calculate the weight of the crown backfill and the granular ballast. The difference between bulk and submerged unit weights is not recognized in the OHBDC (1991).

The majority of the backfill and granular ballast is submerged and the corresponding RFs were calculated for the submerged unit weights and not the saturated unit weights because there is little uncertainty in the unit weight of water. Also, the use of RFs calculated for saturated unit weights can result in negative values for submerged unit weights, which is physically impossible. For example, a design RF of 0.65 was calculated for saturated backfill unit weight having an average value of 13.3 kN/m^3 and a coefficient of variation of 0.191. The factored submerged unit weight would be $(0.65)(13.3\ kN/m^3 - 9.81\ kN/m^3) = -1.1\ kN/m^3$. In comparison, the factored submerged backfill unit weight using the RF (RF = 0.2) for submerged backfill, $(0.2)(13.3\ kN/m^3 - 9.81\ kN/m^3) = 0.70\ kN/m^3$, clearly a more reasonable value. Similar concerns were raised by Boden (1981) who felt that general use of prescribed or probabilistic factors could define soils that could not possibly exist in nature.

The RFs were applied to the mean value for each variable as determined from the measured values, with the exception of the backfill unit weight. The RF for the backfill was applied to the measured backfill unit weights. This was a direct result of how the spreadsheet model was set up (Section 5.2.3.1). This was done to give credit to the areas of the aqueduct where more mineral soils were available for backfill. This is not consistent with the intended use RFs calculated with Equations 2-7 and 2-9 which incorporate a bias factor to account for the difference between the mean value and the nominal value used in design. The effect was negated when selecting the design depth of ballast to be placed over the aqueduct. Details of the effect of this procedure will be shown later.

The same spreadsheet format used for the previous buoyancy analysis was used. Because of the general similarity of the spreadsheets, only the sheets for the analysis using the PSFs from the OHBDC (1991) has been included in this thesis (Appendix 5-1). The others can be provided by the author on request. The changes made to the analysis and spreadsheet are summarized in Table 5-7.

5.2.4.2 Results of Analysis

Figure 5-8 shows a profile of the resisting ratio (Equation 5-8) calculated for each aqueduct segment, with and without granular ballast. The erratic pattern of the profiles is directly related to the variable backfill depths, which are illustrated on Figure 4-2. The large spikes in the profile indicate areas having relatively high backfill unit weights. These spikes also correspond to the change in soil profile shown on Figure 4-1. Some of the less pronounced deviations in the profile are a result of changing aqueduct sections as illustrated by the aqueduct weight profile in Figure 5-8.

Minimum and design depths of granular ballast are shown in Figure 5-9. In general, the area from Mile 85.7 to Mile 88.5 requires additional weight. As mentioned previously, the area west of Mile 85.7 was improved by assuming the groundwater levels could be lowered to the aqueduct crown. A small amount of granular ballast is still required near Mile 85.1 due to a minimal depth of existing backfill. The profile of the design ballast depth was established by taking a more global view of the area, recognizing that there is still some uncertainty in the various parameters used in the buoyancy model. The objectives in establishing the design profile for the ballast were to ensure the majority of the peaks were covered and to provide a reasonably uniform depth of ballast that would be straightforward to construct. The risk associated with not placing ballast over the short areas where the resisting ratio is greater than 1 was deemed to be quite high relative to the cost of placing ballast. In addition, it would be difficult to delineate the exact limits of these areas.

5.2.4.3 Comparison of Results

The results from the two buoyancy analyses described in Sections 5.2.3 and 5.2.4 are compared in Figures 5-10 and 5-11. The results from the analysis completed using the PSFs from the OHBDC (1991) are referenced on the figures as OHDBC and the results from the analysis using project-specific PSFs are referenced on the figures as P-S. The following comparison of results represents original work completed by the author for this thesis.

Figure 5-10 shows the resisting ratio profile from each analysis. In general, the areas having a resisting ratio less than 1.0 are coincident with each other. The largest differences are coincident with areas having higher backfill unit weights. These differences in resisting ratio are directly related to the different RF values used for backfill unit weights.

Figure 5-11 shows a profile of the minimum depths of granular ballast required to provide a resisting ratio of 1.0. In general, the depths determined from each analysis are within 50 to 100 millimetres of each other. The largest differences in depth (250 to 500 millimetres) occur between Mile 87.25 and Mile 88.5. These differences are attributed to the variability of the different parameters and the different RFs used to account for the uncertainty, particularly with respect to backfill unit weight.

As mentioned at the start of this section, the depths of granular ballast calculated from the buoyancy analysis using the PSFs from the OHBDC (1991) (Section 5.2.3) raised concerns about the strength of the aqueduct invert, especially the non-reinforced inverts like the BE invert section shown on Figure 3-14. In general, the use of project-specific PSFs reduced the required ballast depths. The reduced ballast depths were more pronounced between Mile 87.25 and Mile 87.75. A finite element structural analysis was completed for the different aqueduct sections. The area of greatest concern, for invert strength, was the Snake Lake Stretch where a larger aqueduct section was constructed (S section). The results indicated that invert strength was less of an issue for the smaller B and R sections constructed in the Boggy River Stretch (Mile 85.0 to 88.5).

Each term in the buoyancy equation (Equation 5-9) is briefly discussed below to indicate the difference that resulted from using different sets of PSFs and design values. The changes reflect how the original buoyancy analysis was altered to reflect the measured variability of the uncertainties in Equation 5-9.

Two changes were made in calculating the backfill weights using the two approaches. The RF for submerged backfill unit weights was decreased from 0.80 to 0.20 and the RF for bulk backfill unit weights was decreased from 0.80 to 0.65. The effect was a decrease in the backfill weight. The magnitude of the change is directly related to the backfill unit weight used for each aqueduct segment. The resulting differences in backfill weight are illustrated in Figure 5-12.

No changes were included in the shear resistance term.

Two differences were made in calculating the aqueduct weight term using the two methods. The design value for concrete unit weight was increased from 22.5 to 23.35 kN/m^3 and the RF was increased from 0.90 to 0.95. The net effect was an increase in the self-weight term of about 4 to 5 kN/m.

Two differences were included in calculations for the granular ballast weight term. The RF for submerged ballast unit weights was increased from 0.80 to 0.85 and the RF for bulk ballast unit weights was increased from 0.80 to 0.95. The general effect was a net increase in the granular ballast weight term. The magnitude of the change is about 1 to 2 kN/m and is directly related to the depth of ballast required.

The net effect on the total resistance (excluding the weight of granular ballast) due to these changes is the net decrease in total resistance, as shown on Figure 5-13. The most pronounced differences occur where the backfill unit weight is high, as shown by spikes in the total resistance profile for the OHBDC (1991) based analysis.

One difference was included in calculations for the buoyancy force. The LF for this term was decreased from 1.05 to 1.03. The general effect was a net decrease in the buoyant force. The magnitude of the change is about 1.3 kN/m and is directly related to the total section area of the aqueduct.

The net effect on the resisting ratio (excluding the effect of granular ballast) due to the changes discussed above is a net decrease in the resisting ratio, as shown on Figure 5-10. The most pronounced differences occur where the backfill unit weight is high, as shown by spikes in the resisting ratio profile for the OHBDC (1991)-based analysis.

The depths of granular ballast selected for final design are shown on Figure 5-14. From Mile 85.0 to about Mile 87.1, the design depths are within 50 millimetres. East of Mile 87.1 From Mile 87.1 to Mile 88.5 the differences in the design depths range from 50 to

250 millimetres. These larger differences can not be attributed to any single parameter; the differences are due to differences in each of the terms of the buoyancy equation (Equation 5-9).

Although the differences between the two analyses methods are not great, it is important to note that the level of safety for this project could not have been reliably assessed using the PSFs from the OHBDC (1991). The level of safety using the project-specific PSFs is more consistent even though the granular ballast depths are less. The most significant differences in level of safety are reflected by the depths of granular ballast from Mile 87.25 to Mile 88.50 (Figure 5-11). The area of biggest concern would be east of Mile 88.0 where very little ballast was required using the PSFs from the OHBDC (1991). The existing level of safety in this area was overestimated by the OHBDC (1991) analysis.

5.2.5 Limit State Design Using Monte Carlo Simulation

Simulation techniques are considered to be a level 2 or level 3 reliability-based method (Section 2.4.4) depending on the amount of data available. Monte Carlo simulation techniques were used for this thesis. Monte Carlo simulation techniques can be used to estimate the probability characteristics of a function (See Section 2.4.4). Monte Carlo simulation consists of randomly selecting values of the basic variables according to their probability characteristics and then using them in the performance function. (Ayyub and McCuen, 1997) This type of analysis can be completed using the same software package @Risk Version 4.5 (Palisade Corporation 2002) that was used to complete the sensitivity analysis described in Section 5.1.3.

This analysis is significantly different than that completed by UMA Engineering Ltd. and is considered to be original work completed by the author for use in this thesis. This type of analysis was undertaken for a number of reasons. The main reason being to verify the results of the analysis described in Section 5.2.4 where project-specific PSFs were calculated and used to account for uncertainties in the parameters used for the buoyancy analysis. It was necessary to verify the results to ensure the target level of safety (Section 5.2.2) was achieved and also to ensure that the method and assumptions used to calculate the PSFs were reasonable. The second reason is to show another method that can be used to determine a level of safety for various classes of engineering problems. Some concerns were raised in Section 2.4.5 that the use of prescribed PSFs, like those used in Section 5.2.3 from the OHBDC (1991), would discourage the use of engineering judgement or developing an understanding of the inherent geologic variability encountered in many classes engineering projects (Fleming, 1981, Semple 1981, Boden, 1981). The use of Monte Carlo simulations requires the engineer to think about the variability in each parameter because it is necessary to define a probability distribution for each parameter or variable in the analysis. Current computer and software technology has made this method of analysis more feasible than in the past.

5.2.5.1 Probability Distributions

The software program used for the simulation also includes the distribution fitting program BestFit (Palisade Corporation 2002a) that can be used to fit probability distribution functions (PDF) to a data set. Probability distributions were fitted to the data collected within the Boggy River Stretch (Mile 85.0 to Mile 88.5) for the following parameters: crown backfill depths, saturated backfill unit weights, bulk backfill unit weights, concrete unit weights, saturated granular ballast unit weights and bulk granular ballast unit weights. Due to the lack of appropriate data for the concrete unit area and total section area, PDFs could not be fitted using the software. These two uncertainties were modeled in the same manner (triangular PDFs) that was used in the sensitivity analysis presented in Section 5.1.3. The maximum, minimum and mean values required for the triangular distributions were determined for each type of aqueduct section consistent with the procedure described in Section 5.1.3. The PDF for shear strength was based on the normal distribution used for shear strength in the sensitivity analysis presented in Section 5.1.3.

Thirty-seven different PDFs are available in BestFit (Palisade Corporation 2002a). The software fits as many of the available PDFs as possible to the data and ranks the PDFs in order of Chi-Squared 'goodness of fit' statistics. With the exception of concrete unit weight and saturated backfill unit weights, the PDFs having the best 'goodness of fit' statistics were not normal or log-normal. A sensitivity analysis was subsequently completed to determine how sensitive the buoyancy model would be to the distribution type. This was completed by running a simulation of 30,000 simulation cycles with the highest ranked PDFs for each variable to determine a baseline probability of failure (P_f) for a resisting ratio of 1.0 and the depth of granular ballast required to achieve the target level of safety or $P_f = 0.011\%$ that was identified as the design objective in Section 5.2.2. Following this step, the PDF for each variable was changed (only one per simulation) to either a normal or log-normal distribution, whichever seemed to best represent the data (particularly at the tails of the distributions). A simulation was run (30,000 simulation cycles) to determine if the change in PDF resulted in a significant change in the baseline values. Following each simulation, the PDF was switched back to the best fit PDF and another variable was switched. The last check involved switching all the variables to either normal or log-normal PDFs and running the simulation. From the baseline model, the P_f for a resisting ratio less than 1.0 was 60.11 percent and the required depth of ballast was 0.93 metres. The lowest values obtained in the sensitivity analysis were: $P_f = 59.83$ percent for a resisting ratio less than 1.0 and 0.91 metres for the required depth of ballast. The highest values obtained in the sensitivity analysis were: $P_f = 60.11$ percent (from the baseline model) for a resisting ratio less than 1.0 and 0.95 metres for the required depth of ballast. The difference between the maximum and minimum values for P_f and ballast depth is about 0.28 percent and 0.04 metres, respectively. These differences can largely be attributed to the variability in sampling between simulations. This variability was demonstrated by performing 10 simulations of 30,000 simulation cycles. The difference between the maximum and minimum values for P_f and ballast depth is about 0.35 percent and 0.05 metres, respectively. This is comparable to the range of values presented above.

The results of these analyses indicated that it was reasonable to assume normal and log-normal distributions for the measured parameters. Graphs of all the PDFs are included in Figures 5-15 to 5-18. The use of normal and log-normal distributions should provide results that are more comparable to the results from the previous buoyancy analysis that utilized project-specific PSFs based on these distributions types.

5.2.5.2 Buoyancy Model Setup

The assumptions presented in Section 5.2.1 regarding groundwater levels and shear resistance were incorporated into the simulation model. However, the buoyancy model for the Monte Carlo simulation could not be efficiently set up using the same spreadsheet format presented in Section 5.2.3. Instead, a model was set up for each of the eight aqueduct section types (BO, BP, BPM, RA, RE, RG, RPM and RRM, Table 3-1) constructed in the Boggy River Stretch. A total of twelve models were generated, eight were used to evaluate the general site conditions (no shear resistance and submerged crown backfill) for each section type, two models were used to evaluate the effect of including shear resistance with the RA and RE sections, and two models were used to evaluate the effect of having bulk crown backfill unit weights with the BO and RE sections.

The simulation models were set up using a spreadsheet, which works with the software program @ Risk Version 4.5 (Palisade 2002). The simulation model (Figure 5-19) differs from the previous spreadsheet models (Sections 5.2.3 and 5.2.4) in a number of ways. First, no PSFs or design values are used to calculate resistances, instead each variable is defined with a PDF which is randomly sampled to obtain a value. Second, the existing aqueduct segments are no longer considered separately, instead each aqueduct section type is evaluated for the conditions found in the Boggy River Stretch. In the previous spreadsheet models, local conditions for backfill depth and backfill unit weights were applied to each aqueduct segment. These two parameters are now considered as variables.

The model has been set up with the following four general categories used in calculating the terms in the general buoyancy equation (Equation 5-1). The components of the simulation model shown on Figure 5-19 are described below.

- Input variables – defined by PDFs
- Fixed values - values specific to each aqueduct section type
- Disturbing force - one equation to evaluate the buoyant force (Equation 5-7)

- Resisting forces – Equations 5-2 to 5-5 are used to evaluate each resisting force (except the granular ballast term) and the total resisting force. The value for each input variable is determined by sampling the PDF for that variable.
- Resisting ratio – Evaluated with Equation 5-8.
- Additional Resisting Force (granular ballast) - this category is used to determine the minimum depth of granular ballast (Equation 5-6) required to increase the resisting ratio to 1.0 and to calculate the new total resisting force and resisting ratio.

Input Variables

The variables for the model include backfill unit weight (saturated or bulk), crown backfill depth, shear strength, compacted backfill depth, concrete unit weight, concrete area, total section area and ballast unit weight (saturated or bulk). Submerged unit weights were calculated by subtracting the unit weight of water from the sampled value of saturated unit weight.

Each of these input variables is defined by a PDF that is sampled once per simulation cycle. Normal or log-normal PDFs were fitted to the sampled (measured) values for each of the following variables: backfill unit weights, crown backfill depth, concrete unit weight and ballast unit weight. Each PDF was truncated to prevent unreasonable values being used in the simulation. Truncation was necessary because normal and log-normal PDFs will extend well beyond the range of values that could physically occur. The truncation points were selected using engineering judgement and knowledge of the site conditions.

For the two simulation models that used shear strength, the PDFs for shear strength and compacted backfill depth were based on the PDFs used in the sensitivity analysis presented in Section 5.1.3. A correlation coefficient of 0.9 was used to relate backfill unit weight, shear strength and compacted backfill depth. This was done to reduce the potential for high shear strength and compacted depth values from being selected when the backfill unit weight is low, that is when the backfill is likely organic in nature.

The triangular PDFs for the concrete area (Figure 5-20) and total section area (Figure 5-21) are based on the results of the sensitivity analysis Section 5.1.3 that was undertaken to estimate reasonable maximum and minimum values for these two parameters. For each aqueduct section type that was analyzed, the maximum and minimum concrete and total section areas were calculated. The maximum and minimum values were used as the extreme values in a triangular PDF with the mean value equal to the design value. The concrete and total section areas were related using a correlation coefficient of 0.77 (Section 5.1.3) A triangular distribution was selected because little is known about the true distribution of these two parameters. However, there is likely some variability in the two parameters and reasonable maximum and minimum values can be estimated based on expected construction tolerances.

Table 5-8 shows a summary of the probability distributions used in the simulation models.

Fixed Values

Three parameters used in the buoyancy analysis have fixed values. Two of these parameters are specific to each aqueduct section type, the maximum aqueduct width which is used in calculating the crown backfill and ballast weights and the arch leg backfill volume which is used to calculate the weight of arch leg backfill. The only other parameter having a fixed value is the unit weight of water (9.81 kN/m^3).

Disturbing and Resisting Forces

The disturbing (buoyant) force and each resisting force were calculated using Equations 5-2 to 5-7. During the simulation, these equations are calculated once per simulation cycle. The value for each variable in the equations was randomly sampled once per simulation cycle from the corresponding PDF.

Resisting Ratio and Additional Resisting Force

The resisting ratio is calculated once per simulation cycle, using the sum of the individual resisting forces and the buoyant force. If the resisting ratio is less than 1.0, a depth of granular ballast is calculated to provide a resisting ratio equal to 1.0. The unit weight of ballast is sampled from the PDF defined for this variable.

Simulation Settings

Thirty-thousand simulation cycles were completed for of the twelve models. This number of simulation cycles was selected because it reduced the time to run the simulation and significantly reduced the size of the output data files while still providing a stable output. This was tested by running 10 simulations of 30,000 simulation cycles and 10 simulations of 100,000 simulation cycles. Table 5-9 shows the maximum, minimum and average values for the probability of the resisting ratio (RR) being less than 1.0 and the range and average of the required ballast depth having a probability of being exceeded equal to 0.011percent. The results in Table 5-9 also show that increasing the number of simulation cycles from 30,000 to 100,000 reduces the difference between the maximum and minimum values for P_f and ballast depth by approximately 50%. That is, the difference between the maximum and minimum values for P_f and ballast depth is reduced by 0.189 percent and 0.02 metres, respectively. From a practical point of view, increasing the number of simulation cycles from 30,000 to 100,000 would not significantly change the depth of ballast chosen for design. In fact, the design ballast depth chosen for the aqueduct section (BO section) used in this sensitivity analysis was 0.8 metres, which is equal to or greater than the maximum value for either scenario. In addition, the true PDFs for the variables are unknown and attempting to achieve a tight convergence interval may imply that the probability of failure is well defined. In fact, the objective is to achieve a target level of safety. The number of simulation cycles used for the work presented in this thesis was 30,000.

5.2.5.3 Simulation Results

The amount of statistical data generated by @ Risk Version 4.5 (Palisade 2002) for each simulation is very considerable and is not included in this thesis. A sample of the output from the BP aqueduct section is provided in Appendix 5-2. This section type was selected because it was the most frequent section type constructed in the Boggy River Stretch. For all the other simulations, a summary of the output statistics and a summary of the simulation results for the resisting ratio, including a histogram, are included in Appendix 5-3.

From the perspective of a buoyancy analysis based on reliability methods, the most important statistics from the simulation include the probability of failure, P_f (defined by resisting ratio, RR < 1) and the minimum depth of ballast required to provide the target level of safety, that is a P_f = 0.011% that RR < 1 (Figures 5-22 and 5-23, respectively). The ballast depths corresponding to a probability of 99.989 percent (100% - 0.011%) and 100.0 percent were taken from the output statistics to help determine a practical design depth. The design depth was subsequently entered into a second simulation model for the respective aqueduct section to determine the corresponding probability of failure (RR < 1) (Figure 5-24).

The simulation results for each aqueduct section type analysed indicate that there is a high probability, typically greater than 30%, that buoyancy will occur if the aqueduct is dewatered. Additional resisting force (granular ballast) is required to reduce the probability of failure to the target level of safety (P_f = 0.011%). A summary of the probabilities of failure and the ballast depths for both simulations (original conditions and design values) are presented in Table 5-10.

5.2.5.4 Comparison of Methods

Figure 5-25 compares the minimum ballast depths calculated from the Monte Carlo simulation and the LSD method using project-specific PSFs (Section 5.2.4). In general, the ballast depths are within 50 to 100 millimetres of each other, indicating that both methods are providing similar levels of safety. Based on the results of the Monte Carlo simulation, this level of safety is comparable to the target level of safety, that is a probability of failure of about 0.011 percent. Some of the differences may be attributable to the rounding down of the calculated RFs to select a design RF (Table 5-6). Since the design RFs are lower, a slightly higher ballast depth would be calculated.

There are some subtle differences in the model results. For example, from Mile 85.0 to Mile 85.5 and from Mile 87.95 to Mile 88.05 the two methods indicate that different ballast depths are required. The ballast depth from the simulation model is about 200 millimetres for the first area and about 600 millimetres for the second area. In contrast, the ballast depths from the PSF method are zero, except for a short piece near Mile 85.1. The difference in ballast depth is directly attributable to the depths of the crown backfill. The simulation cycles with resisting ratios less than 1.0 also had depths of backfill that were less than 0.4 metres. However, in these two areas, the measured depth of backfill over the aqueduct segments is typically greater than 0.65 metres. Unexpected results from the Monte Carlo simulation can be checked by reviewing the values used in each simulation cycle. The PSF model utilized these measured depths of backfill that resulted in sufficient backfill weight to keep the resisting ratio greater than 1.0, that is no ballast was required. The conclusion drawn is that the Monte Carlo simulation does a good job at modeling the global conditions but the buoyancy model using PSFs better represents local conditions at each aqueduct segment. This is not a result of the PSFs used in the analysis but in how the two models were set up. The PSF model analyzed each aqueduct segment and the simulation model analyzed each aqueduct section type.

The similarity of the results also indicates that the assumptions made when calculating the PSFs in Section 5.2.4 are reasonable. These assumptions include: statistical independence between all variables (a requirement for Equations 2-6 to 2-9), the value of the separation coefficient (0.6), the value of the safety index (3.5) and the value of the LF applied to the buoyant force.

5.2.6 Working Stress Design

For comparative purposes and to demonstrate the level of uncertainty in the working stress design (WSD) method, a fourth buoyancy analysis was completed using this method. This analysis is considered original work completed by the author for use in this thesis. It will be remembered that WSD only requires a single global factor of safety (GFS). The buoyancy model used for the WSD is simply a copy of the spreadsheet model used for the PSF analysis in Section 5.2.4. To convert this model to WSD format, all the PSFs were set to 1.0, the design values for concrete unit weight, undrained shear strength and granular ballast unit weight remained at the same value. The need for granular ballast was evaluated for each aqueduct segment with the following equation:

[5-10] (Total Resisting Force)(GFS) $\geq$ Buoyant Force

If the factored total resistance was less than the buoyant force a depth of granular ballast was calculated to provide the additional resisting force.

The biggest uncertainty here is the value of GFS that should be used. This question was raised in discussion with several experienced engineers. The responses are summarized below:

- A GFS of 1.1 could be used if the weight of the backfill is ignored in the stability calculation. That is, only the weight of the aqueduct and the buoyant force are considered.
- A GFS of about 1.3 could be used if the backfill weight is included in the stability calculation.
- One engineer commented that a value of 1.5 would be tried first and if the required depth of granular ballast seemed unreasonably high a lower GFS would be used.

There is obviously a lot of variability in the responses. Two questions still remain, what GFS should be used? and what will the probability of failure be? The only way to estimate the probability of failure would be to use a simulation model like that shown in the previous section.

The WSD model was run for a family of safety factors ranging from 1.0 to 1.5. A GFS of 1.25 almost mirrors the ballast depth profile from the PSF method where the PSFs from the OHBDC (1991) were used (Figure 5-26). This is not a total coincidence because the most common RF used in that analysis was 0.80 for which the inverse value is 1.25. The ballast profile required for a GFS of 1.20 is about 100 millimetres less than the ballast profile for a GFS of 1.25. This indicates that for an increase or decrease in the GFS of 0.05 the required depth of ballast will change by about 100 millimetres. In practice, it is expected that the value of the GFS would be increased or decreased in increments of 0.1. This would amount to an incremental change of 200 millimetres for granular ballast depth.

5.2.7 Discussion

A considerable amount of additional work has been completed for the purpose of this thesis to gain a larger understanding of limit states and reliability-based design processes and their applicability to many classes of engineering problems. A listing of the main components of additional work completed for this thesis is provided below.

- A parameter sensitivity analysis was completed using basic information that would have been available at the time in the Buoyancy Assessment Program when the decision was made to determine project-specific PSFs.
- The calculation of the project-specific PSFs has been re-done to reflect the shape of the probability distributions.
- A sensitivity analysis was completed to determine the influence of each parameter in the equations used to calculate the PSFs, and to verify that the assumptions made regarding these parameters were reasonable.
- The variability in concrete area and total section size was incorporated into the analysis.
- Significantly more work was completed to interpret the results from each of the analyses.
- A more comprehensive simulation model was developed to verify the analysis results obtained using project-specific PSFs and to determine if the target level of safety was provided.
- A WSD analysis was completed to show the potential variability in the analysis and selection of design values.

A number of objectives were identified in Section 1.3. They are:

1. To show that LSD methods can be successfully applied to non-codified engineering problems.
2. To show why PSFs from non-related design codes may not be applicable.
3. To show how project-specific PSFs can be developed.
4. To show how reliability-based design methods can be used as an alternative to the PSF method and to check the results of designs completed with project-specific PSFs.

5. To demonstrate the inherent uncertainty in using WSD methods.
6. To demonstrate the importance of engineering judgement in design.

The buoyancy analysis work completed in this chapter demonstrates how well LSD methods can be adapted to non-codified engineering problems. The remedial repairs to address buoyancy concerns were completed by September 2001 following which the aqueduct was shutdown for a period of about 3 weeks, no signs of the development of buoyancy were observed.

Although the buoyancy analysis completed using the PSFs from the OHBDC (1991) were not significantly different than the results obtained using project-specific PSFs the value of the PSFs calculated were significantly different than those in the OHBDC (1991), as shown in Table 5-7. The thought process and calculations used in calculating the project-specific PSFs is well documented in Section 5.2.4.

A Monte Carlo simulation was completed to show an alternative limit state or reliability-based design method that can be used to directly evaluate the target level of safety. The design depths of granular ballast determined with this method were similar to those for the two PSF methods indicating they are all providing a similar level of safety.

The potential uncertainty in the level of safety using WSD methods is obvious based on the discussion in the preceding section and also in the discussion below. The rational approach used for the LSD methods is well demonstrated in this chapter. When comparing the work required to evaluate the PSFs and the PDFs used in the different analyses for this project, the lack of rational thought in selecting a single GFS based on engineering experience is obvious. The rational process used for the LSD methods provides a quantifiable target level of safety and provides a good understanding of the variability involved.

A considerable amount of engineering judgement was used in the LSD-based analyses and should be evident as the reader advances through Chapter 5. Examples of the engineering judgement required include, among others, the delineation of a groundwater profile along the aqueduct, estimating the variability in concrete area and total section size based on knowledge of original construction practice and some measured values of concrete thickness and internal dimensions, the selection of parameters for use in the shear resistance term of Equation 5-9, the interpretation of the various sensitivity analysis results and the interpretation of the data measured for the various parameters in the buoyancy model.

All four of the methods used for the buoyancy analysis can provide reasonably similar results. It would be interesting to ask a group of four engineers to complete the buoyancy analysis using the same information but with each engineer using one of the four different methods presented in this thesis. The engineers using the three LSD methods would likely produce similar results because they would be using rational methods to determine the PSFs and the design values. However, the engineer using the WSD method could quite easily design a ballast depth profile at least 100 millimetres above or below the ballast depth profiles obtained from the LSD methods. This is how much the design depth could vary by changing the GFS by +/- 0.05. If a GFS of much more than 1.3 was chosen, the level of safety would be greater than that provided by the LSD results but would be considerably less economical. It could also lead to additional hazard of structural failure of the aqueduct. This was a very important aspect of the work completed, because additional resisting forces were required to reduce the risk of buoyancy but the structural capacity of the non-reinforced aqueduct was limited. However, if a GFS much less than 1.25 was used the level of safety drops quite rapidly. What are the odds that an engineer would have chosen a GFS of 1.25? In any case, the level of safety could not be quantified using the WSD method.

The three limit state design methods produced similar results. The least rational method is the one which used the PSFs from the OHBDC (1991). It is the least rational of the three methods because no consideration was given to the actual variability of the different uncertainties in the buoyancy model. The potential for error is large when using PSFs developed for other purposes and having different dispersion statistics.

The other two limit state methods (the LSD method using project-specific PSFs and the Monte Carlo simulation method) accounted for the uncertainties in rational ways. It is noted however, that the probability distributions assumed for the various parameters may not be normally or log-normally distributed and that this forms a limiting set of assumptions in the use of these methods, at least for the calculation of the project-specific PSFs because that is one of the assumptions made in deriving Equations 2-6 to 2-9. It is very likely that one or more of the parameters fitted these distributions reasonably well. The error involved with selecting an inappropriate probability distribution for a variable diminishes with the number of variables in the model. Others may have viewed the data differently and may have selected other probability distributions, this is a subjective matter that can not be standardized.

The Monte Carlo simulation method allows more flexibility from the point of view of being able to select a number of different probability distributions. These distributions can also be truncated to prevent unreasonable values from being sampled and used in an analysis. In addition, the different parameters can be correlated in the event that they are not statistically independent. This is not possible when using PSFs since that method assumes that the variables are statistically independent. The Monte Carlo method also allows sensitivity analyses to be completed and this may help in making better engineering decisions.

The results of the four different analyses are broadly comparable but the level of confidence in the results may vary considerably. The LSD method with project-specific PSFs and the Monte Carlo method provide the best indication of the level of safety because they directly account for uncertainties in each of the variables.

6.0 CONCLUSIONS

6.1 INTRODUCTION

The work presented in this thesis was undertaken to show that limit states design (LSD) and reliability-based design methods can be used to solve engineering problems that lie outside the scope of current design codes. This proposition was successfully tested through the use of an actual design case that assessed the potential for buoyancy of the Shoal Lake Aqueduct when it was dewatered for maintenance repairs. Three different LSD methods were used, together with measured data and engineering judgement, to evaluate the buoyancy potential of the aqueduct. These results were compared with a WSD analysis to show the degree of uncertainty associated with this latter method.

A number of objectives were listed in Chapter 1 and reviewed in Chapter 5. Results of the buoyancy analysis show that these objectives have been met.

Chapter 1 proposed the following hypothesis. "A combination of limit states design (including reliability-based design methods) and good engineering judgement can provide technically sound, economic solutions to many classes of engineering problems, particularly problems involving remediation of existing infrastructure." The following conclusions can be made.

6.2 CONCLUSIONS

1. Engineering judgement is an essential component of design, no matter what design method is used.
2. Limit states design using reliability methods provide a rational means of evaluating different limit states, partial safety factors, design values and comparing the relative benefits of different designs.
3. Simulation models (such as the Monte Carlo Method) provide more flexibility than the partial safety factor method especially when combining various parameters, provided sufficient data are available.

4. Reliability-based designs produce increased confidence that limit states will not be exceeded, resulting in more reliable performance, and/or decreased project costs.

6.3 RECOMMENDATIONS FOR FURTHER WORK

The work undertaken in this thesis focussed on buoyancy assessment of the Shoal Lake Aqueduct. The work has shown that LSD methods can be applied to this problem. Additional work is required to confirm and promote the use of LSD to geotechnical engineers. These recommendations are summarized below.

1. Development of geotechnical models that are consistent with LSD methodology in that they evaluate ultimate limit states using strength equations and evaluate serviceability limit states using strain or deformation equations.
2. Development of site characterization procedures that will provide more a consistent level of site information.
3. Development of experience-based databases for geotechnical parameters that can be used to supplement site investigation results.
4. Education of engineering students and practicing engineers as to the application of LSD methods and to show how uncertainties in design can be accounted for using probabilistic and reliability-based methods.

REFERENCES

AASHTO, 1994. AASHTO LRFD bridge design specifications, First edition, American Association of State Highway and Transportation Officials.

Alen, C. and Jendeby, L., 1993. Design value of the bearing capacity for a friction pile at a given risk level. *In* Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, May 26-28. *Sponsored by* the Danish geotechnical Society. Vol. 1, pp. 91-100.

Allen, D.E., 1975. Limit states design – a probabilistic study. Canadian Journal of Civil Engineering, **2**: 36-49.

Ayyub, B.M. and McCuen, R.H., 1997. Probability, statistics, and reliability for engineers. CRC Press LLC, New York.

Baecher, G.B. and Christian, J.T., 2000. Natural variation, limited knowledge, and the nature of uncertainty in risk analysis. 25th Anniversary Engineering Foundation Conference on Risk and Uncertainty in Water Resources Engineering, Santa Barbara, October 2000.

Becker, D.E., 1996a. Eighteenth canadian geotechnical colloquium: limit states design for foundations. Part I. An overview of the foundation design process. Canadian Geotechnical Journal, **33**: 956-983.

Becker, D.E., 1996b. Eighteenth canadian geotechnical colloquium: limit states design for foundations. Part II. Development for the National Building Code of Canada. Canadian Geotechnical Journal, **33**: 984-1007.

Becker, D.E., Burwash, W.J., Montgomery, R.A. and Liu, Y., 1998. Foundation design aspects of the Confederation Bridge. Canadian Geotechnical Journal, **35**: 750-768.

Bengtsson P.E. et al., 1993. A comparative study on limit state design and total safety design for shallow foundations. *In* Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, May 26-28. *Sponsored by* the Danish Geotechnical Society. Vol. 1, pp. 13-22.

Boden, B., 1981. Limit state principles in geotechnics. Ground Engineering, **14** (6): 2-7.

Bolton, M.D., 1981. Limit state design in geotechnical engineering. Ground Engineering, **14** (6): 39-46.

Bolton, M.D., 1993. What are partial factors for?. *In* Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, May 26-28. *Sponsored by* the Danish geotechnical Society. Vol. 3, pp. 565-583.

Bowles, J.E., 1968. Foundation analysis and design, First edition, McGraw-Hill Inc., New York, NY.

CEN, 1992. Geotechnical design, general rules. european committee for standardization (CEN), Eurocode 7. Danish Geotechnical Institute, Copenhagen.

CH2M Gore and Storrie, 1998. Shoal lake aqueduct condition assessment report for the city of winnipeg water and waste department (draft#1).

Chase, W.G., 1920a. Construction features of the water works of the Greater Winnipeg Water District. Proceedings, Annual American Water Works Association Convention, Montreal, Canada, June 1920. City of Winnipeg, Water and Waste Department, Resource Centre.

Chase, W.G., 1920b. Notes and instructions for the guidance of the staff responsible for the care and operation of the aqueduct supplying water to the communities comprising the Greater Winnipeg Water District. City of Winnipeg, Water and Waste Department, Resource Centre.

City of Winnipeg, 1965. Report on shoal lake aqueduct. City of Winnipeg, Water and Waste Department, Resource Centre.

CSA, 2000. Canadian highway bridge design code (CHBDC). Standard CAN/CSA-S6-00, Canadian Standards Association, Rexdale, Ontario.

Day, R.A., 1997. Structural limit states design procedures in geomechanics. Bridging the millenia, Proceedings of the Ausroads 1997 Bridge Conference, Sydney, G.J. Chirgwin ed. **1**, 275-286.

Day, R.A., 1998. Limit states design in geotechnical engineering – consistency, confidence or confusion?. Unpublished.

Fleming, W.G.K., 1989. Limit state in soil mechanics and the use of partial factors. Ground Engineering, **22** (7): 34-35.

Fuertes, J.H., 1920. The basic principles used in the designs for the new water supply works of Winnipeg, Manitoba. Proceedings, Annual American Water Works Association Convention, Montreal, Canada, June 1920.

Gore and Storrie, 1995. Buoyancy assessment program. Working paper no. 1 (draft), phase 1. Buoyancy assessment from mile 82 to intake. Assessment and rehabilitation of the shoal lake aqueduct.

Green, R. and Becker, D.E., 2000. National report on limit state design in geotechnical engineering: Canada. LSD2000: International Workshop on Limit State design in Geotechnical Engineering, Melbourne, Australia. November 18, 2000.

Griffin, D. and Morgan, D., 1998. - A new water projection model accounts for water efficiency. Western Canada Water and Wastewater Association Conference, Calgary, Alberta, October 1998.

GWWD, 1907. Report on a new water supply for the city of winnipeg manitoba. City of Winnipeg, Water and Waste Department, Resource Centre.

GWWD, 1914. Contract Numbers 30, 31, 32, 33, and 34. Greater Winnipeg Water District. For construction of aqueduct from Shoal Lake. City of Winnipeg, Water and Waste Department, Resource Centre.

GWWD, 1917. The development of the plan for supplying water to the various municipalities forming the Greater Winnipeg Water District, City of Winnipeg, Water and Waste Department, Resource Centre.

GWWD, 1918. Aqueduct construction scheme, what it is, what it means. City of Winnipeg, Water and Waste Department, Resource Centre.

GWWD, 1919. Drawing P-180 – Contract 34, Profile of aqueduct as constructed. City of Winnipeg, Water and Waste Department.

MacGregor, J.G., 1975. Safety and limit states design for reinforced concrete. 1978-79 National Lecture Tour. Sponsored by the Structural Division of the Canadian Society for Civil Engineering and the Portland Cement Association.

Manitoba Department of Energy and Mines. Surficial Geological Map of Manitoba, Map 81-1.

Meyerhof, G.G., 1984. Safety factors and limit states analysis in geotechnical engineering. Canadian Geotechnical Journal, **21**: 1-7.

Meyerhof, G.G., 1995. Development of geotechnical limit state design. Canadian Geotechnical Journal, **32**: 128-136.

Ministry of Transportation of Ontario, 1983. Ontario Highway Bridge Design Code (OHBDC). 2nd ed., Downsview, Ont. Two vols.

Ministry of Transportation of Ontario, 1991. Ontario Highway Bridge Design Code (OHBDC). 3rd ed., Downsview, Ont. Two vols.

Morgenstern, N.R., 1995. Managing risk in geotechnical engineering. Proceedings of Xth Panamerican Conference on Soil Mechanics and Foundation Engineering, Guadalajara, Mexico, Vol. 4, 1996.

NRC, 1975. National building code of canada (NBCC). National Research Council of Canada, Ottawa, Ontario.

NRC, 1995. National building code of canada (NBCC). National Research Council of Canada, Ottawa, Ontario.

Oliphant, J., 1993. A proposed limit state design approach for geotechnical design. *In* Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, May 26-28. *Sponsored by* the Danish geotechnical Society. Vol. 1, pp. 41-49.

Orr, T.L.L. and Farrell, E.R., 2000. Geotechnical design to Eurocode 7. Springer-Verlag London Limited 1999.

Ovesen, N.K., 1981. Towards a european code for foundation engineering. Ground Engineering, **14** (7): 25-28.

Palisade Corporation, 2002. @Risk Version 4.5 Professional. Risk analysis and simulation add-in for Microsoft Excel, Version 4.5. Newfield, New York.

Palisade Corporation, 2002a. BestFit. Probability distribution fitting software, add-in for Microsoft Excel. Newfield, New York.

Phoon, K.-K., Kulhawy, F.K. and Grigoriu, M.D. , 1993. Observations on reliability-based design of foundations for electrical transmission line structures. *In* Proceedings of the International Symposium on Limit State Design in Geotechnical Engineering, Copenhagen, May 26-28. *Sponsored by* the Danish Geotechnical Society. Vol. 2, pp. 351-362.

Phoon, K.-K. and Kulhawy, F.H., 1999. Characterization of geotechnical variability. Canadian Geotechnical Journal, **36**: 612-624.

Ravindra, M.K., and Galambos, T.V., 1978. Load and resistance factor design for steel. Journal of the Structural Division, ASCE, Vol. 104, No. ST9, September 1978, pp. 1337-1353.

Semple R.M., 1981. Partial coefficient design in geotechnics. Ground Engineering, **14** (6): 47-48.

Simpson, B., Pappin, J.W. and Croft, D.D., 1981. An approach to limit state calculations in geotechnics. Ground Engineering, **14** (6): 21-26.

UMA Engineering, 2000. Assessment and rehabilitation of the shoal lake aqueduct. Buoyancy assessment program. Geotechnical investigation Mile 85 to Mile 95. Working paper 1B.

UNIES Ltd., 1998. GWWD aqueduct project – Boggy river flood potential along right of way, mile 85-89.

UNIES Ltd., 1999. GWWD aqueduct project – Snake lake buoyancy assessment, mile 93.5 to 97.5.

Vesic, F., 1971, Breakout resistance of objects embedded in ocean bottom. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM9, September 1971, pp. 1183-1205.

TABLES

Table 2-1. Approximate relationship between safety index and probability of failure.
(MacGregor 1975 and Becker 1996b)

Safety Index, β	Probability of Failure (log-normal distribution)
1.0	0.159
1.28	0.100
1.64	0.050
2.00	0.023
2.32	0.010
3.0	1.35×10^{-3}
3.5	1.1×10^{-4}
4.0	3.2×10^{-5}
4.5	3.3×10^{-6}

Table 3-1. Summary of aqueduct sections constructed between Mile 85.0 and the Intake.

Section Type*	Internal Dimensions Height (metres)	Internal Dimensions Width (metres)	Inside Area (metres²)	Concrete Thickness Invert (millimetres)	Concrete Thickness Crown (millimetres)	Concrete Area Invert (metres²)	Concrete Area Arch (metres²)	Concrete Area Total (metres²)	Total Section Area (metres²)	Invert Construction reinforced with steel	Invert Construction extended mm	Area Below Haunch Grade (metres³/m)	Maximum External Width (metres)	Maximum External Height (metres)	Years Constructed	Comments
BA	2.251	2.667	4.636	254	152	0.888	1.327	2.215	6.851	yes	0	0.923	3.658	2.657	1917, 1918	reinforced invert
BO	2.251	2.667	4.636	152	152	0.617	1.325	1.942	6.578	no	0	0.650	3.658	2.556	1915, 1917, 1918	standard section
BP	2.251	2.667	4.636	152	152	0.803	1.325	2.127	6.763	in extension	203	0.483	4.064	2.556	1916	widened invert
BPM	2.251	2.667	4.636	152	152	0.825	1.460	2.285	6.921	in extension	203	0.483	4.064	2.556	1916	widened invert, thicker arch
BRxing	2.251	2.667	4.636	203	238	0.913	2.059	2.972	7.608	yes	0	0.922	4.343	2.692	all	road crossing
B piled	2.251	2.667	4.636	356	152	1.783	1.337	3.120	7.756	yes	0	n/a	3.962	2.759	all	wood pile foundation
BE	2.251	2.667	4.636	152	152	0.707	1.325	2.032	6.668	in extension	305	0.391	4.251	2.556	1917, 1918	widened invert
RPM	2.276	2.699	4.775	152	162	0.823	1.507	2.330	7.106	in extension	203	0.512	4.096	2.591	1916	widened invert, thicker arch
RRM	2.276	2.699	4.775	292	162	1.202	1.507	2.709	7.484	yes	203	0.890	4.096	2.731	1916	reinforced invert, thicker arch
RA	2.276	2.699	4.775	254	162	0.895	1.327	2.222	6.998	yes	0	0.903	3.689	2.692	1917, 1918	reinforced invert
RA piled	2.276	2.699	4.775	305	162	1.698	1.334	3.033	7.808	yes	0	n/a	3.988	2.743	1917, 1918	wood pile foundation
RE	2.276	2.699	4.775	152	162	0.715	1.327	2.042	6.817	in extension	279	0.397	4.280	2.591	1917, 1918	widened invert
RO	2.276	2.699	4.775	152	162	0.625	1.327	1.952	6.727	no	0	0.632	3.683	2.591	1915, 1917, 1918	standard section
RG	2.276	2.699	4.775	635	162	2.609	1.327	3.936	8.711	yes	0	0.298	4.267	3.073	all	gravity invert
RR xing	2.276	2.699	4.775	203	238	1.003	2.094	3.098	7.873	yes	0	0.899	4.267	2.718	all	road crossing
SP	2.743	3.277	7.014	165	178	0.991	1.989	2.980	9.994	in extension	203	0.740	4.775	3.086	1916	widened invert
SR	2.743	3.277	7.014	368	178	1.671	1.989	3.660	10.674	yes	203	1.420	4.775	3.289	1916	reinforced invert
SG	2.743	3.277	7.014	787	178	3.805	1.989	5.794	12.809	yes	0	0.695	4.369	3.708	all	gravity invert
SA	2.743	3.277	7.014	311	178	1.297	1.989	3.286	10.300	yes	0	1.382	4.369	3.232	1917, 1918	reinforced invert
Section	2.743	3.277	7.014	165	178	0.863	1.989	2.852	9.866	in extension	279	0.628	4.959	3.086	1917, 1918	widened invert
SO	2.743	3.277	7.014	165	178	0.800	1.989	2.789	9.804	no	0	0.886	4.369	3.086	1915, 1917, 1918	standard section
SR-xing	2.743	3.277	7.014	229	279	1.931	3.218	5.150	12.164	yes	0	1.550	5.372	3.251	all	road crossing
Ry-xing	2.743	3.277	7.014	521	305	3.512	3.010	6.522	13.536	yes	0	n/a	5.334	3.569	all	railway crossing

***NOTE:**
- first letter of section type refers to the internal size of the pipe
- second letter of section type refers to the style of invert used.
- third letter of section type refers to a slightly thickened arch.
- 'B' section constructed between Mile 85.0 and Mile 86.98, L = 3,186 metres
- 'R' section constructed between Mile 86.98 and Mile 89.09, L = 3,395 metres
- 'S' section constructed between Mile 89.09 and Mile 97.517 (Intake), L = 13,562 metres

Table 4-1. Summary of forces for 60 percent submergence of the aqueduct. Assumes no backfill weight or backfill shear resistance.

Section Type*	Aqueduct Self-Weight, W (kN/m)	Buoyant Force, B (kN/m)	Ratio Of Forces, (W/B)
BO	45.3	47.8	0.95
RO	45.6	46.5	0.98
SO	65.1	65.8	0.99

*Three general section sizes used between Mile 85 and the Intake at Mile 97.5.

Table 4-2. Summary of measured concrete core thicknesses.

Core No.	Aqueduct Mileage	Section Type	Thickness (millimetres)		
			Measured	Design	Difference
S232	95.680	SE	276	183	94
S362-a	94.585	SE	229	177	52
S362-b	94.585	SE	191	177	14
S700-a	91.746	SE	236	165	71
S700-b	91.746	SE	141	165	-24
S700-c	91.746	SE	185	165	20
S701	91.738	SE	170	165	5
S923	89.853	SR	191	178	13
R130	88.009	RE	278	287	-9
R129	88.009	RE	179	177	2
R213-a	87.295	RRM	355	287	68
R213-b	87.295	RRM	571	244	327*
B1047-a	86.595	BP	301	299	2
B1047-b	86.595	BP	234	244	-10
B1159-a	85.643	BO	322	299	23
B1159-b	85.643	BO	159	177	-18
B1162-a	85.617	BO	315	305	10
B1162-b	85.617	BO	177	177	0
				Average*	18.4
				Maximum*	94
				Minimum	-24

*Note: measurement difference for core no. R213-b not included in statistics.

Table 4-3. Summary of measured internal dimensions. Figure 4-14 shows the measurement locations inside the aqueduct.

	Inside Height	Inside Width	Diagonal (d1)	Diagonal (d2)
	(millimetres)		(millimetres)	
S – Section				
Design Value	2743	3242	3297	3297
Measurements	338	338	338	338
Average	2747	3244	3288	3286
Difference From Design	+4	+2	-9	-11
Standard Deviation	14.7	24.9	19.8	18.8
Coefficient of Variation	0.00535	0.00767	0.0060	0.0057
R – Section				
Design Value	2276	2670	2693	2693
Measurements	55	55	55	55
Average	2283	2688	2698	2700
Difference From Design	+7	+18	+5	+7
Standard Deviation	10.7	13.5	15.5	16.2
Coefficient of Variation	0.00469	0.00502	0.00574	0.006
B – Section				
Design Value	2251	2637	2659	2659
Measurements	50	50	50	50
Average	2257	2666	2671	2673
Difference From Design	+6	+29	+12	+14
Standard Deviation	8.4	10.2	12.4	15.2
Coefficient of Variation	0.00372	0.00382	0.00464	0.00568

Table 5-1. Probability distributions used in parameter sensitivity analysis.

Parameter	Distribution Type	Mean	Std. Dev.	Max.	Min.
$\gamma_{sat.\ backfill}$ (kN/m^3)	Normal	15.5	3.0	23.0	10.0
Crown backfill depth (m)	Normal	0.9	0.3	1.5	0.2
Shear strength (kPa)	Normal	21.0	11.4	50.0	0.0
Compacted backfill depth (m)	Normal	0.9	0.4	1.2	0.0
$\gamma_{concrete}$ (kN/m^3)	Normal	22.5	0.79	25.0	20.0
Concrete area (m^2)	Triangular	1.94	N/A	2.33	1.56
Total section area (m^2)	Triangular	6.58	N/A	6.79	6.38

Table 5-2. BO aqueduct section - Sensitivity analysis results for concrete area and total section area.

Case	Total Section Area	Inside Area	Concrete Area	Aqueduct Weight		Buoyant Force		Resisting Ratio, RR		Comments
				$\gamma_{concrete}$ =	23.35	γ_{water} =	9.81	RR = self wt/buoy force		
	(m^2)	(m^2)	(m^2)	(kN/m)	%of design	(kN/m)	%of design	RR	%of design	
A	6.58	4.64	1.94	45.35	1.00	64.53	1.00	0.70	1.00	dimensions as per design
B	6.79	4.63	2.16	50.37	1.11	66.56	1.03	0.76	1.08	inside area as per design, outside shell offset 25.4 mm outward
C	6.38	4.64	1.73	40.49	0.89	62.56	0.97	0.65	0.92	inside area as per design, outside shell offset 25.4 mm inward
D	6.58	4.81	1.77	41.30	0.91	64.53	1.00	0.64	0.91	outside area as per design, inside shell offset 25.4 mm outward
E	6.58	4.46	2.12	49.39	1.09	64.53	1.00	0.77	1.09	outside area as per design, inside shell offset 25.4 mm inward
F	6.79	4.45	2.33	54.41	1.20	66.56	1.03	0.82	1.16	outside shell offset 25.4mm outward, inside shell offset 25.4mm inward
G	6.38	4.82	1.56	36.44	0.80	62.56	0.97	0.58	0.83	outside shell offset 25.4mm inward, inside shell offset 25.4mm outward
H	6.38	4.47	1.91	44.54	0.98	62.56	0.97	0.71	1.01	both shells offset inward 25.4mm
I	6.79	4.80	1.98	46.32	1.02	66.56	1.03	0.70	0.99	both shells offset outward 25.4mm

Note: calculated correlation coefficient between total section area and concrete area = 0.77

Table 5-3. Regression and correlation analysis results for the parameter sensitivity analysis. The results indicate how strong the input variables are related to the resisting ratio.

Input Parameter	Regression Coefficient		Correlation Coefficient	
	(with shear)	(no shear)	(with shear)	(no shear)
Sq. of Correlation Coeff. (R^2)	0.980	0.981	N/A	N/A
$\gamma_{sat. backfill}$ (kN/m^3)	0.465	0.927	0.959	0.935
Shear strength (kPa)	0.427	N/A	0.952	N/A
Crown backfill depth (m)	0.131	0.302	0.117	0.284
Compacted backfill depth (m)	0.119	N/A	0.943	N/A
Concrete area (m^2)	0.079	0.181	0.058	0.141
$\gamma_{concrete}$ (kN/m^3)	0.035	0.079	0.038	0.072
Total section area (m^2)	-0.033	-0.052	0.035	0.081

Table 5-4. Statistics for the input and output variables from the parameter sensitivity analysis.

	Min.	Max.	Mean	Std. Dev.	Coeff.Of Variation
Input Variables					
$\gamma_{\text{sat. backfill}}$ (kN/m^3)	10.00	22.99	15.68	2.70	0.172
Crown backfill depth (m)	0.20	1.50	0.89	0.27	0.303
Shear strength (kPa)	0	49.98	21.68	10.29	0.474
Compacted backfill depth (m)	0	1.20	0.76	0.28	0.368
γ_{concrete} (kN/m^3)	20.00	24.99	22.50	0.78	0.035
Concrete area (m^2)	1.56	2.33	1.94	0.16	0.082
Total section area (m^2)	6.38	6.79	6.58	0.08	0.012
Output (Forces)					
Crown backfill weight (kN/m)	0.19	69.98	19.13	10.88	0.569
Arch leg backfill weight (kN/m)	0.65	45.09	20.07	9.23	0.460
Shear resistance (kN/m)	0	119.95	37.94	26.80	0.706
Aqueduct weight (kN/m)	32.59	56.36	43.73	3.86	0.088
Total resisting force (kN/m)	36.55	275.39	120.86	44.48	0.368
Resisting ratio	0.57	4.27	1.87	0.69	0.369

Table 5-5. Partial safety factors obtained from the OHBDC (1991).

Variable	PSF Symbol (Equation 5-9)	PSF
Backfill weight	$\phi_{backfill}$	0.80*
Backfill shear resistance	ϕ_{shear}	0.50
Aqueduct weight	$\phi_{concrete}$	0.90*
Granular ballast weight	$\phi_{ballast}$	0.80*
Buoyant force	α	1.05*

*From OHBDC Third Ed., Table 2-5.1 (b)
ϕ = resistance factor
α = load factor

Table 5-6. Project-specific partial safety factors.

Variable	Coefficient Of Variation	Probability Distribution	PSF Calculated	PSF Design
$\gamma_{Concrete}$	0.017	Normal	$\phi = 0.965$	$\phi = 0.95$
$\gamma'_{submerged\ ballast}$	0.054	Normal	$\phi = 0.887$	$\phi = 0.85$
$\gamma_{bulk\ ballast}$	0.024	Normal	$\phi = 0.949$	$\phi = 0.95$
$\gamma'_{submerged\ backfill}$	0.728	Log-normal	$\phi = 0.217$	$\phi = 0.20$
$\gamma_{bulk\ backfill}$	0.191	Log-normal	$\phi = 0.670$	$\phi = 0.65$
Shear strength	N/A	N/A	$\phi = 0.50$*	$\phi = 0.50$*
Buoyant force	N/A	N/A	$\alpha = 1.05$*	$\alpha = 1.03$*

PSF based on engineering judgement
ϕ = resistance factor
α = load factor

Table 5-7. Summary of the buoyancy analysis parameters used with the partial safety factor methods.

	Analysis	OHBDC		Project-Specific	
Variable	Units	Mean/Design Value	PSF	Mean/Design Value	PSF
$\gamma_{Concrete}$	(kN/m^3)	22.50	$\phi = 0.90$	23.35	$\phi = 0.95$
$\gamma'_{submerged\ ballast}$	(kN/m^3)	8.19	$\phi = 0.80$	8.19	$\phi = 0.85$
$\gamma_{bulk\ ballast}$	(kN/m^3)	14.40	$\phi = 0.80$	14.40	$\phi = 0.95$
$\gamma'_{submerged\ backfill}$	(kN/m^3)	Variable	$\phi = 0.80$	Variable	$\phi = 0.20$
$\gamma_{bulk\ backfill}$	(kN/m^3)	Variable	$\phi = 0.80$	Variable	$\phi = 0.65$
Shear Strength	(kPa)	12.0	$\phi = 0.50$	12.0	$\phi = 0.50$
Buoyant Force (total section area)	(m^3)	As per design	$\alpha = 1.05$	As per design	$\alpha = 1.03$

ϕ = resistance factor
α = load factor

Table 5-8. Probability distributions of the input parameters used in the monte carlo simulation.

Parameter	Distribution Type	Mean	Std. Dev.	Min	Max
$\gamma_{sat.\ backfill}$ (kN/m^3)	Log-normal	13.32	2.92	10.0	20.0
Crown backfill depth (m)	Normal	1.08	0.27	0.15	2.0
Shear strength (kPa)	Normal	21.0	11.4	10.0	40.0
Compacted backfill depth (m)	Normal	0.9	0.4	0.30	1.20
$\gamma_{concrete}$ (kN/m^3)	Normal	23.35	0.39	21.0	25.0
Concrete area (m^2)	Triangular	1.94	N/A	2.33	1.56
Total section area (m^2)	Triangular	6.58	N/A	6.79	6.38
$\gamma_{bulk\ ballast}$ (kN/m^3)	Normal	14.36	0.35	12.3	16.3
$\gamma_{sat.\ ballast}$ (kN/m^3)	Normal	18.02	0.45	16.0	20.0

Table 5-9. Variation in the simulation results using 30,000 and 100,000 simulation cycles. Summary of probabilities for a resisting ratio less than 1.0 and the corresponding depth of ballast required to provide a resisting ratio of 1.0.

	30,000 simulation cycles		100,000 simulation cycles	
	P_f (%) that RR<1.0	Ballast Depth (m)	P_f (%) that RR<1.0	Ballast Depth (m)
Maximum	49.760	0.801	49.726	0.794
Minimum	49.406	0.746	49.561	0.760
Difference	0.354	0.055	0.165	0.034
Average	49.630	0.772	49.618	0.776

Table 5-10. Monte carlo simulation results – Initial simulation shows the probability of failure (RR<1.0) that the resisting ratio is less than 1.0 and the minimum depths of ballast required to provide a resisting ratio of 1.0. The final simulation results show the probability of failure (RR<1.0) for the design depth of ballast.

Aqueduct Section	Initial Simulation (original conditions)			Final Simulation (design values)	
	RR*<1	Minimum Ballast Depth (m)		Design Ballast Depth (m)	RR* < 1
	P_f (%)	P =99.989%	P=100%		P_f (%)
BO	49.56	0.76	0.78	0.80	0.007
BP	33.56	0.61	0.64	0.65	0.003
BPM	27.82	0.57	0.61	0.60	0.007
RA	38.48	0.72	0.75	0.75	<0.011
RE	36.59	0.63	0.65	0.65	0.007
RG	0.143	0.09	0.13	N/A	**
RPM	29.60	0.58	0.60	0.60	0.010
RRM	12.83	0.49	0.53	0.50	0.003
BO-bulk*	0.19	0.15	0.21	0.20	<0.011
RA-shear*	5.04	0.48	0.54	0.55	<0.011
RE-bulk*	0.067	0.07	0.10	N/A	**
RE-shear*	5.82	0.46	0.51	0.50	<0.011

*Notes: BO-bulk denotes model where bulk crown backfill weights are used

RA-shear denotes model where shear resistance included.

RR = resisting ratio

** - see Section 5.2.5.3 for additional information.

FIGURES

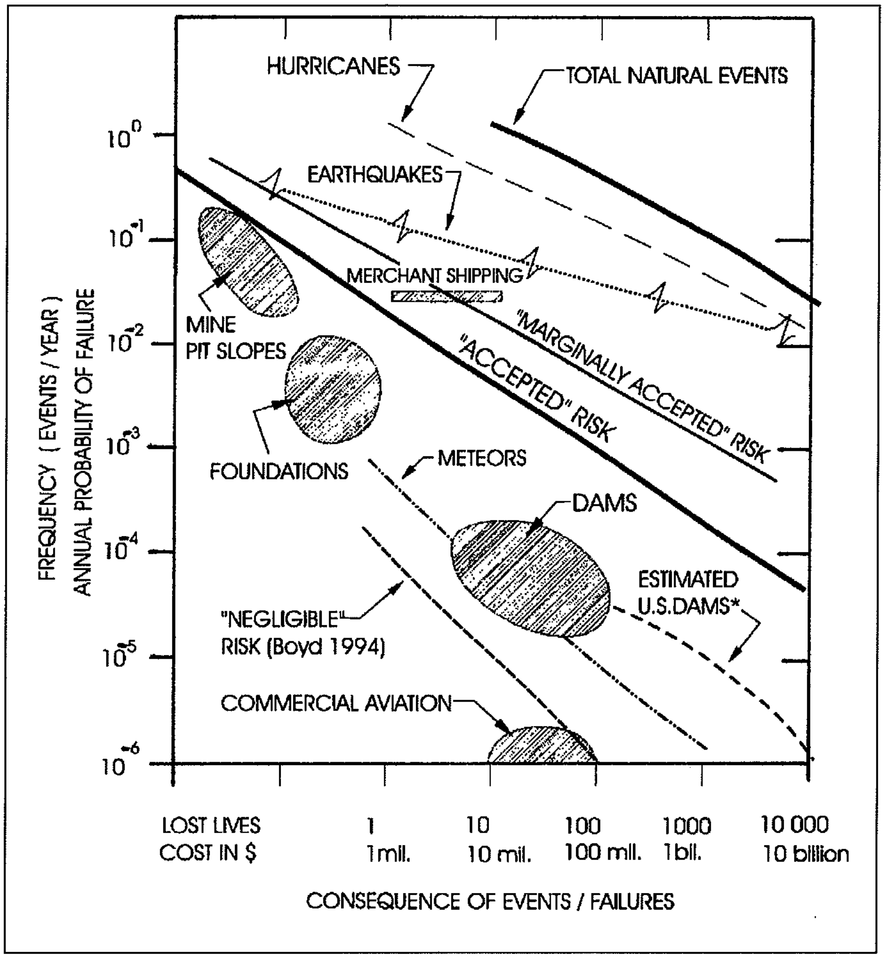


Figure 2-1 Risks for selected natural events and engineering projects designed in keeping with current practice. (from Becker 1996a)

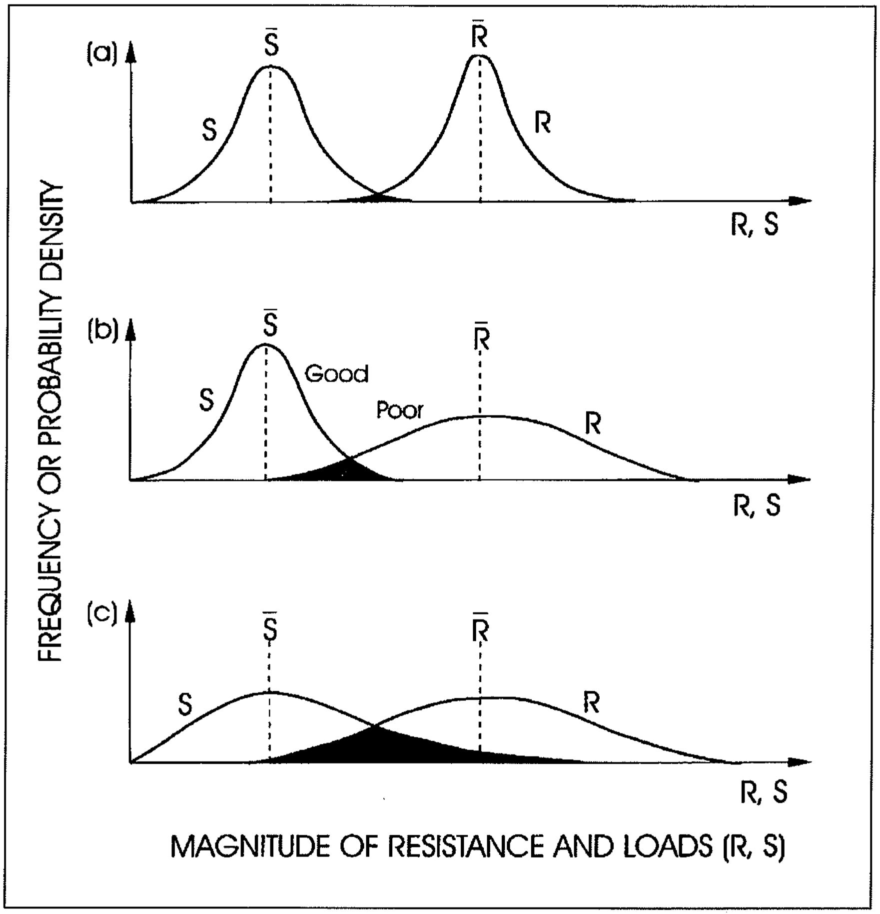


Figure 2-2. Possible load and resistance distributions (after Green 1989): (a) very good control of R and S; (b) mixed control of R and S; (c) poor control of R and S. (Becker 1996b)

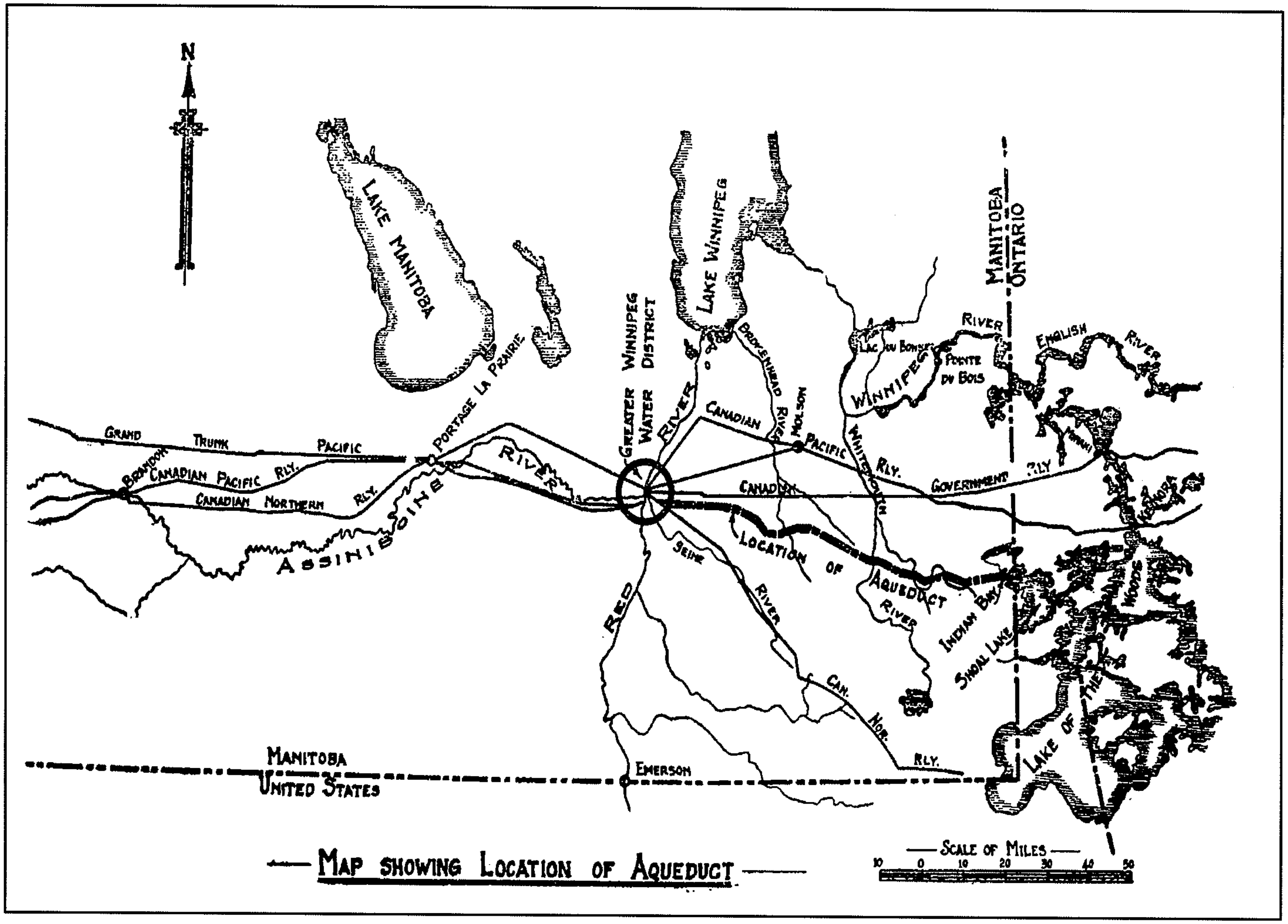


Figure 3-1. Map showing the location of Winnipeg, Indian Bay and the aqueduct.

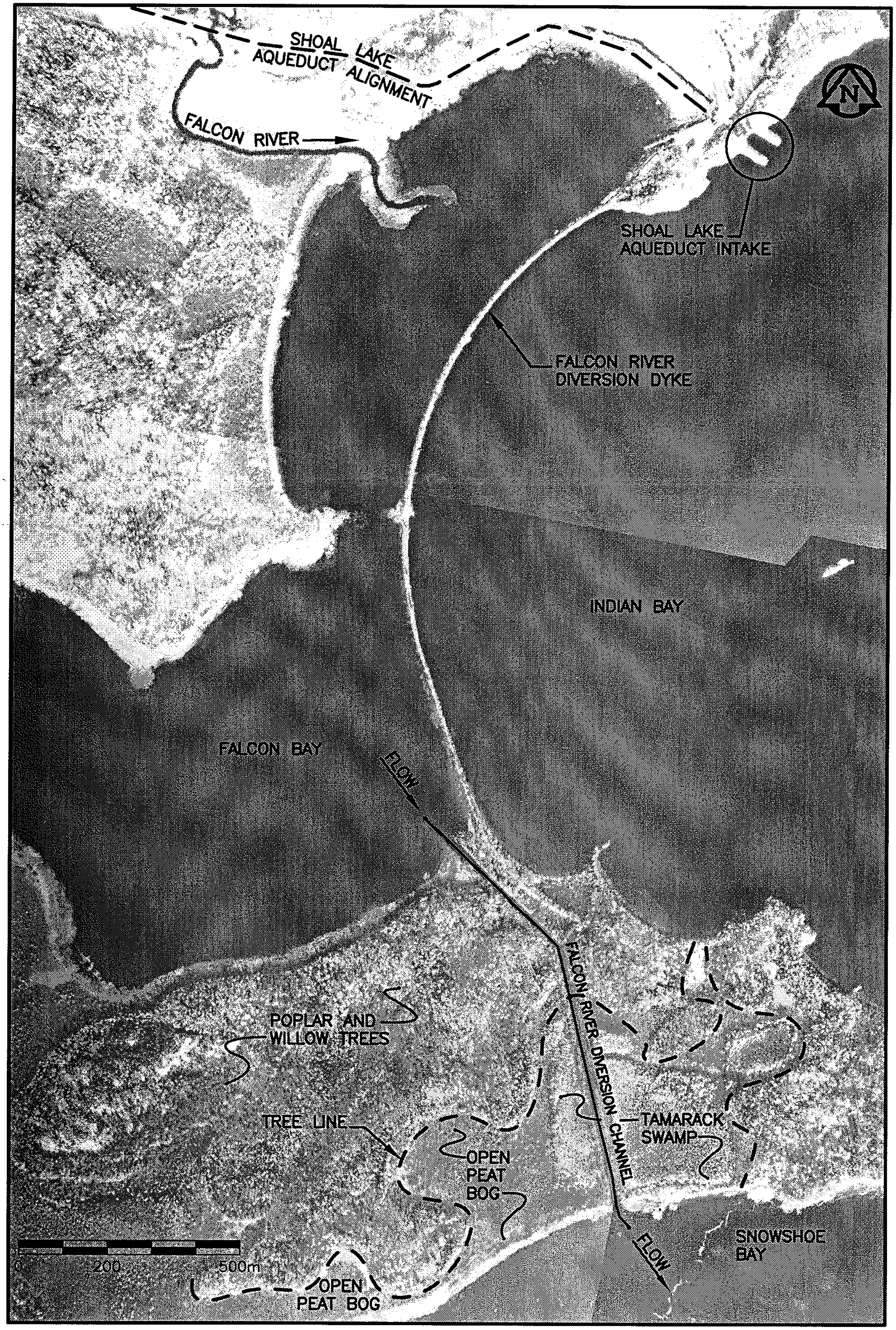


Figure 3-2. Falcon river diversion.

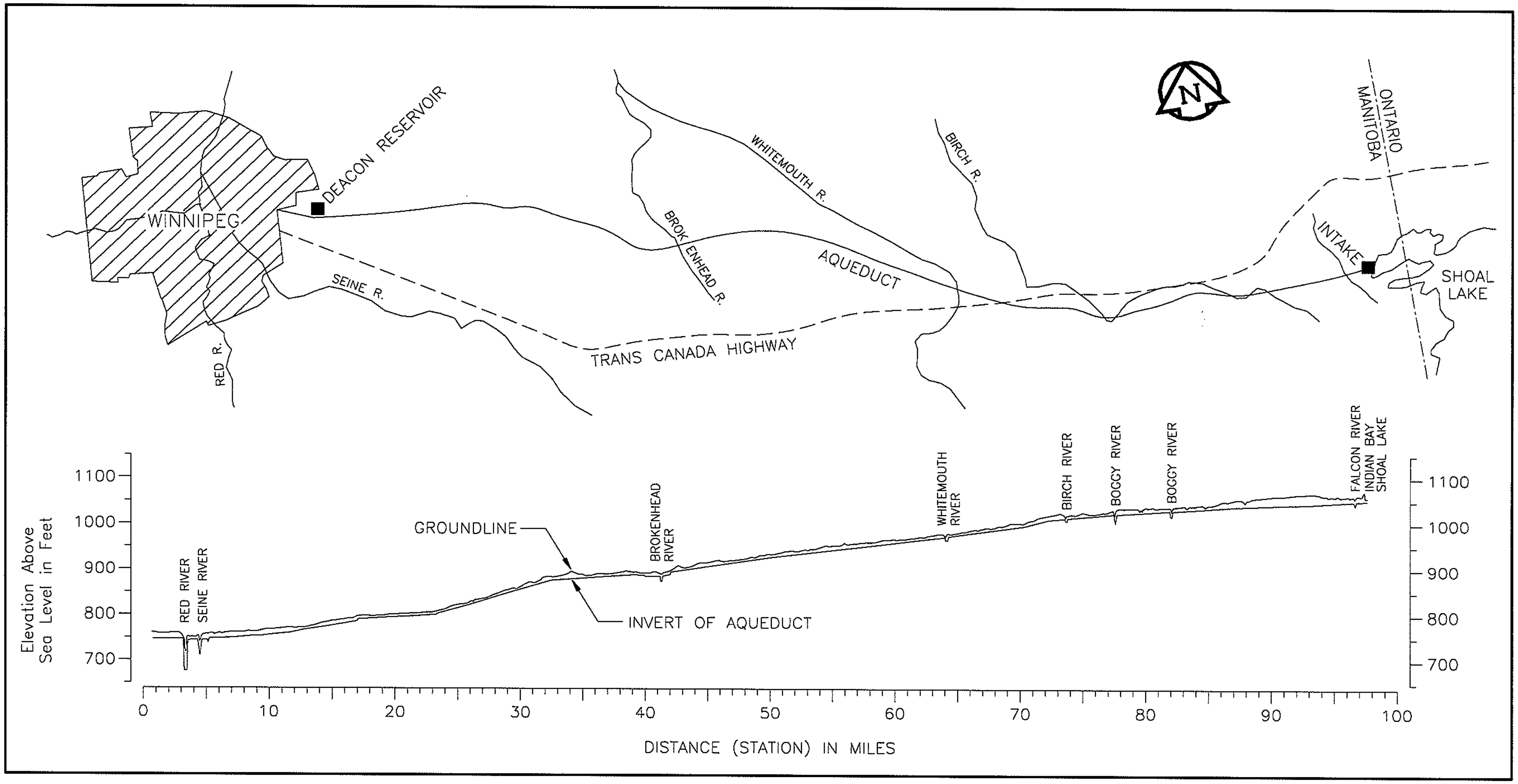


Figure 3-3. Plan and profile of the aqueduct alignment.

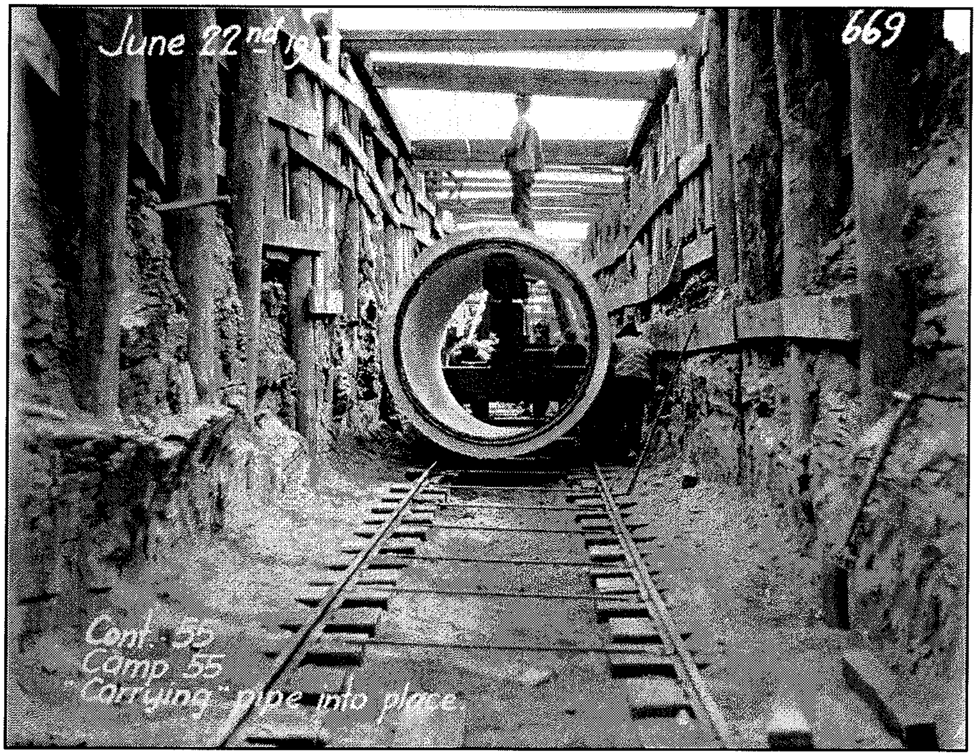


Figure 3-4. Pre-cast pipe segment in shored trench. (Construction Photograph No. 669)

Figure 3-5. Cut and cover construction. (Construction Photograph No. 552)

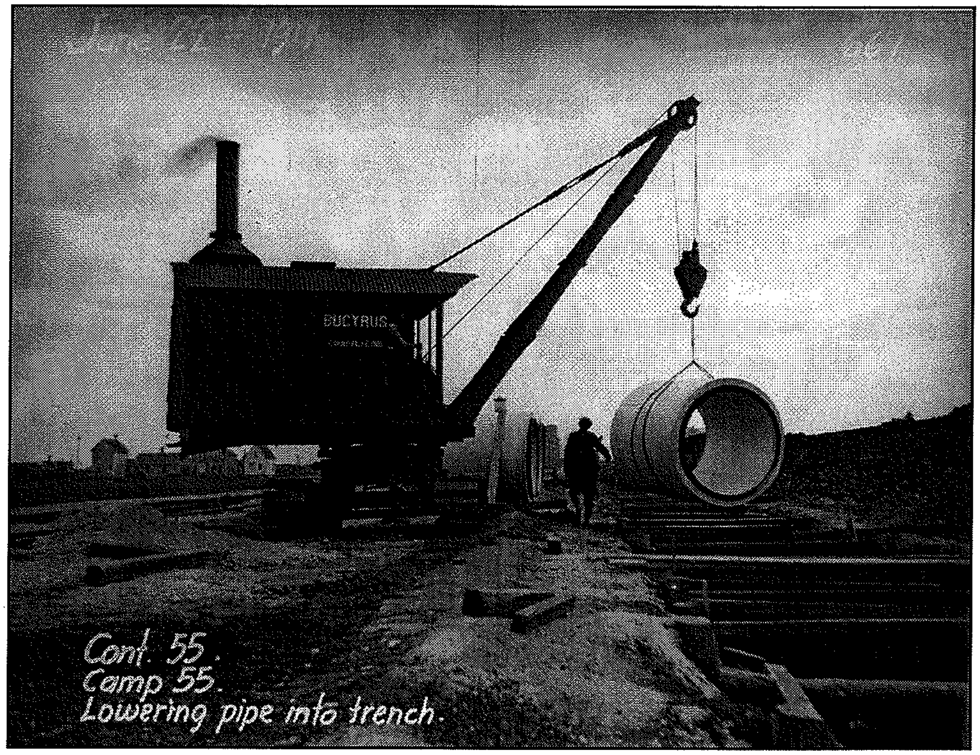


Figure 3-6. Pre-cast pipe segment used to construct the Branch 1 Aqueduct. (Construction Photograph No. 667)

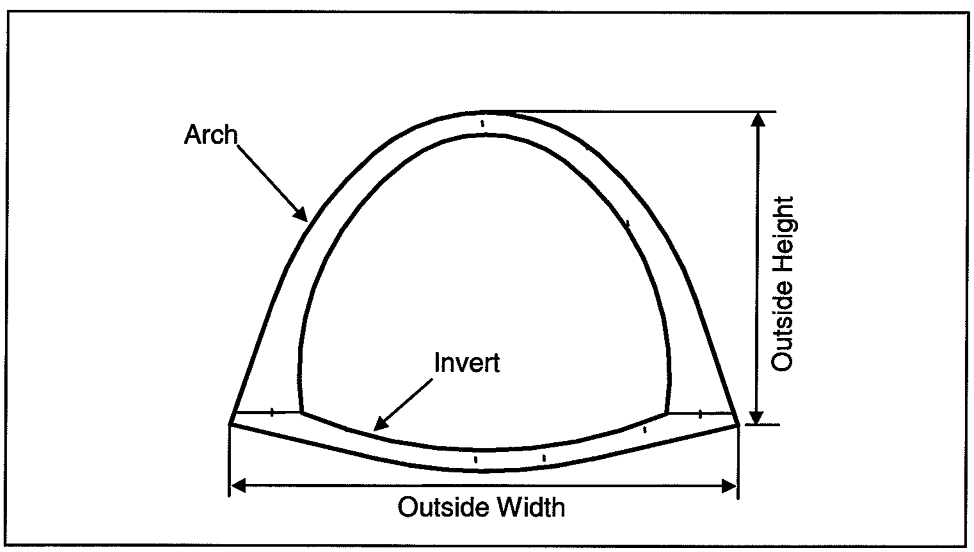


Figure 3-7. Aqueduct section consisting of an arch and invert.

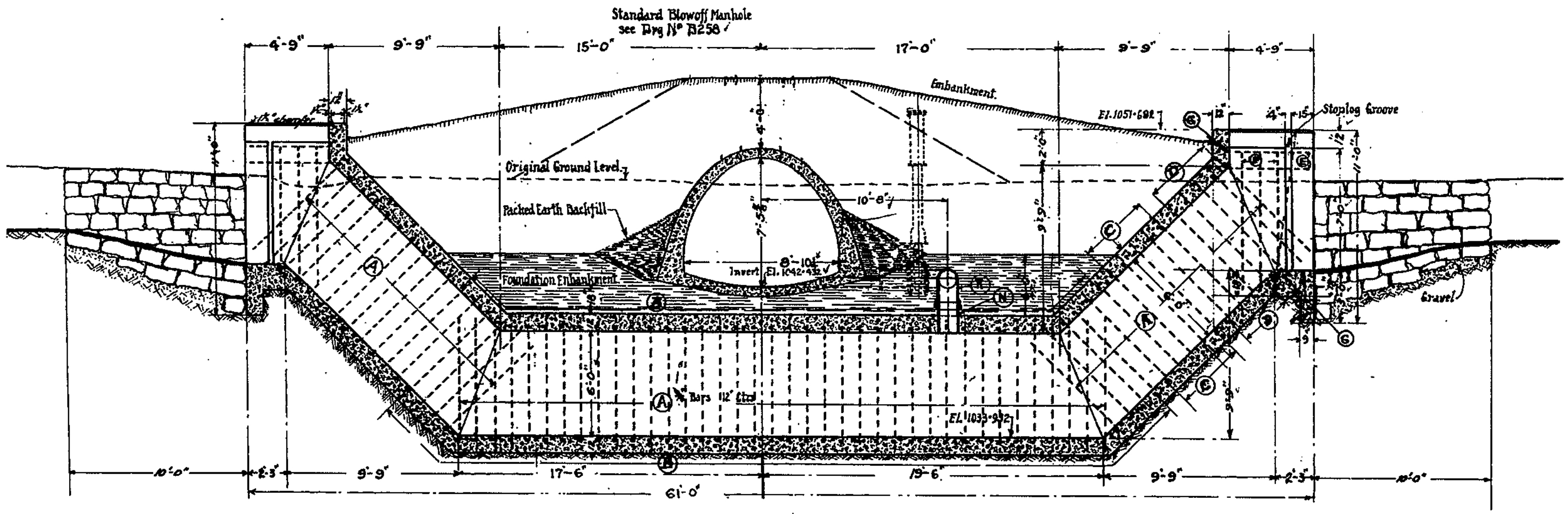


Figure 3-8. Inverted drainage siphon used to maintain drainage paths perpendicular to the alignment of the aqueduct. (From GWWD Drawing D-327)

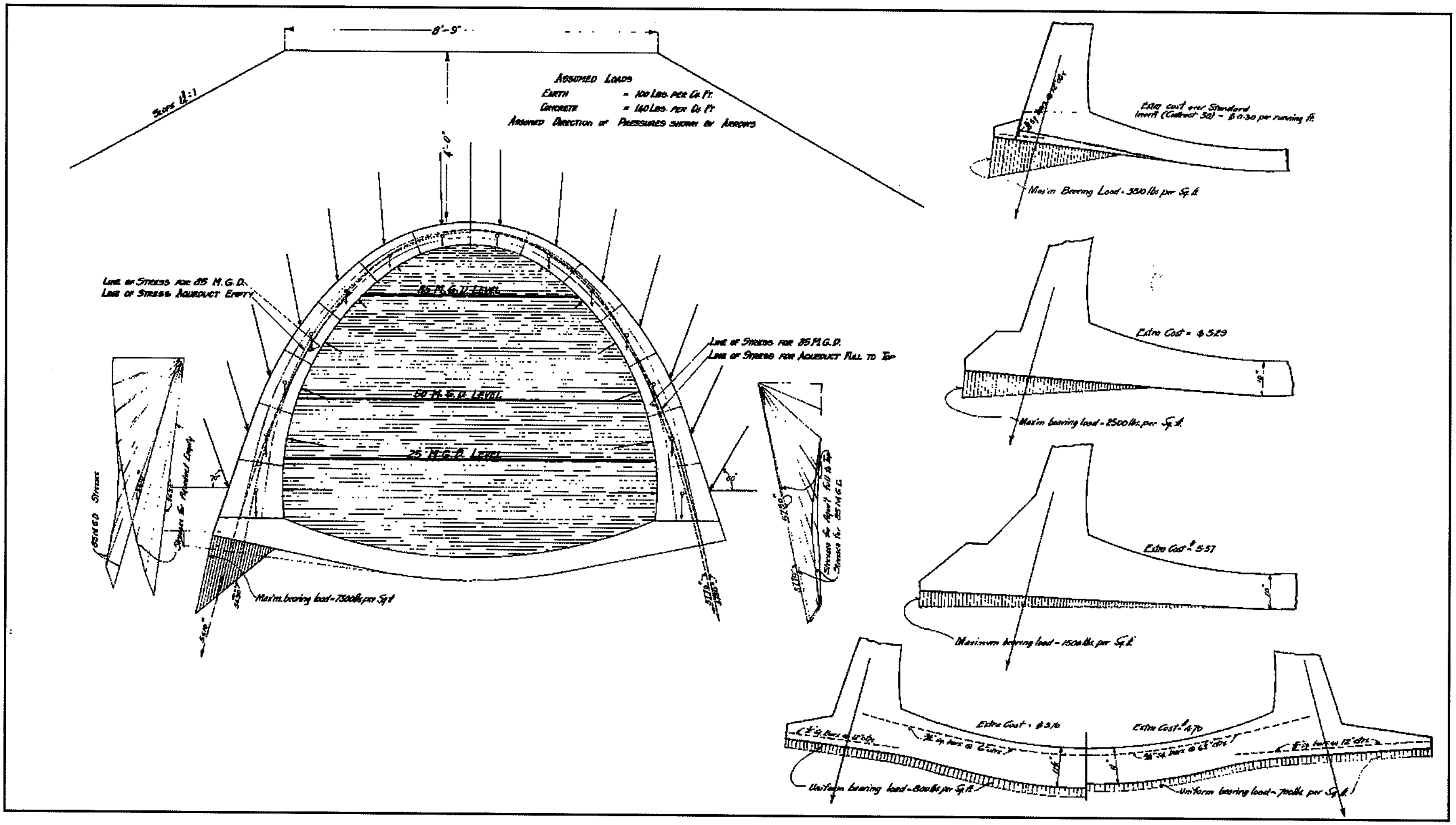


Figure 3-9. Typical arch stresses and bearing loads for the aqueduct. (from GWWD Drawing D317)

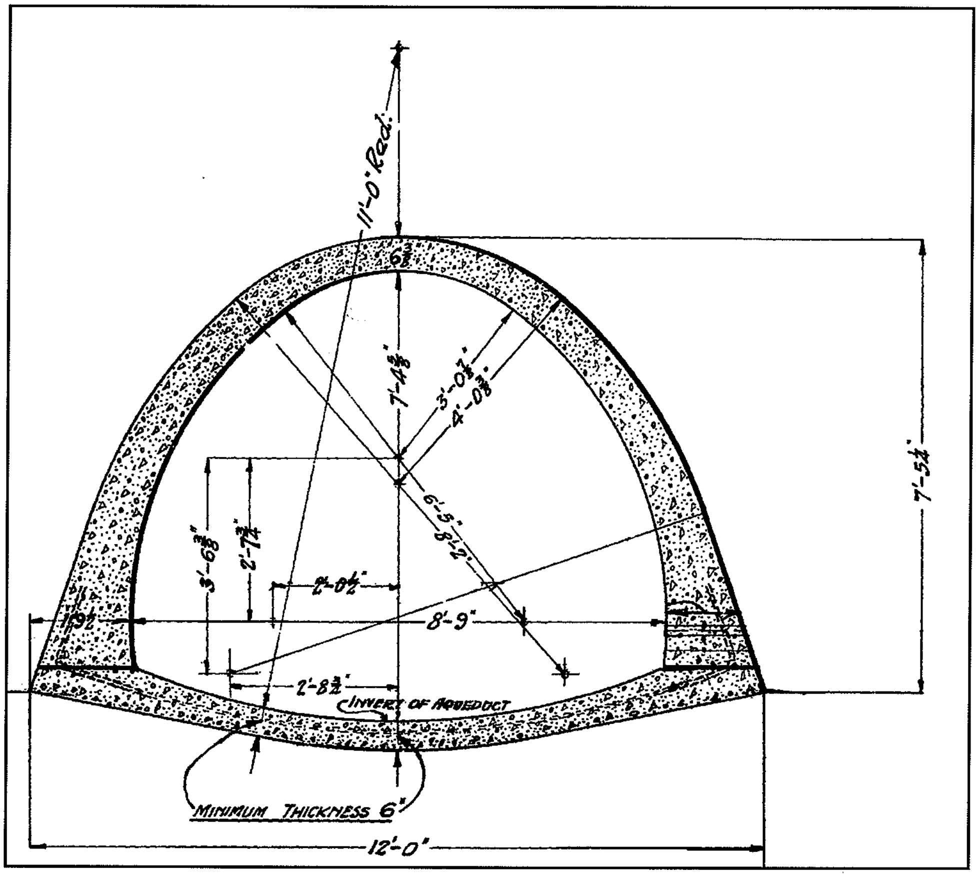


Figure 3-10. Typical aqueduct section used in 1915. The invert is 150 millimetres thick and is non-reinforced. The invert section shown is also referred to as the standard or 'O' type invert. (from GWWD Drawing B-128)

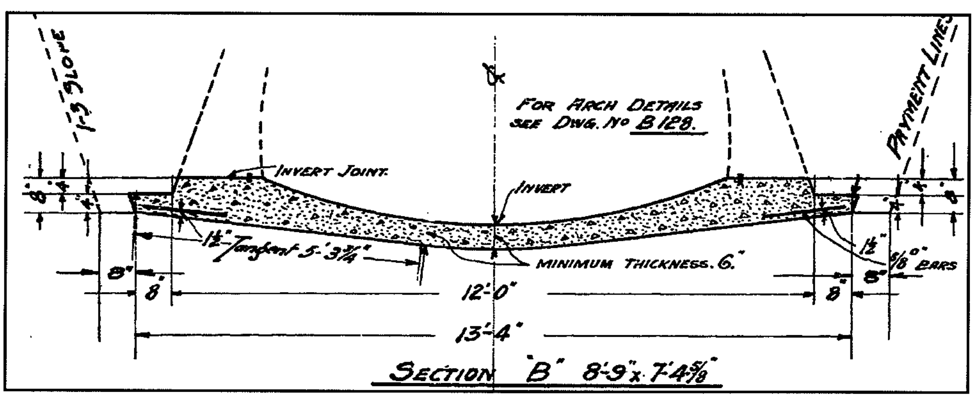


Figure 3-11. Widened invert slab, used in 1916, to reduce the foundation bearing pressure. This invert type was used on all solid foundations, whether compressible or not. (from GWWD Drawing D-334)

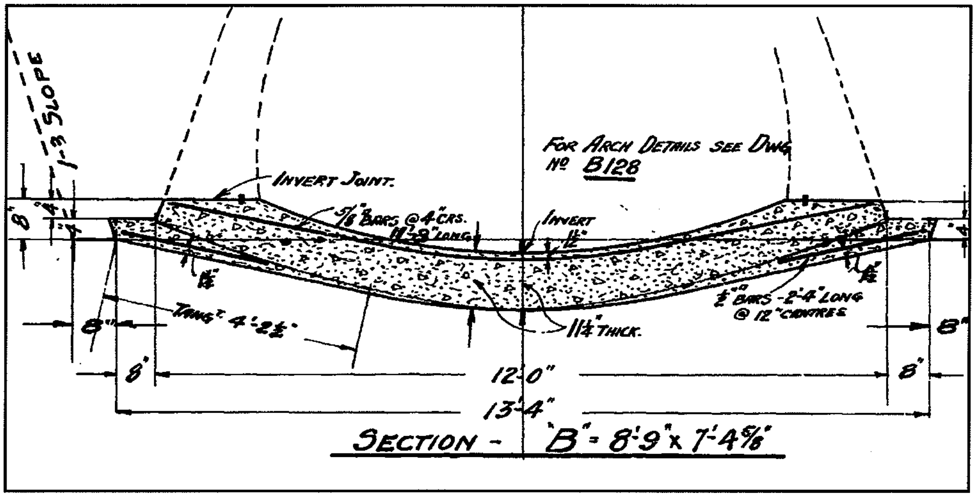


Figure 3-12. Widened and reinforced invert slab used in 1916 on all questionable foundation soils. (from GWWD Drawing D-334)

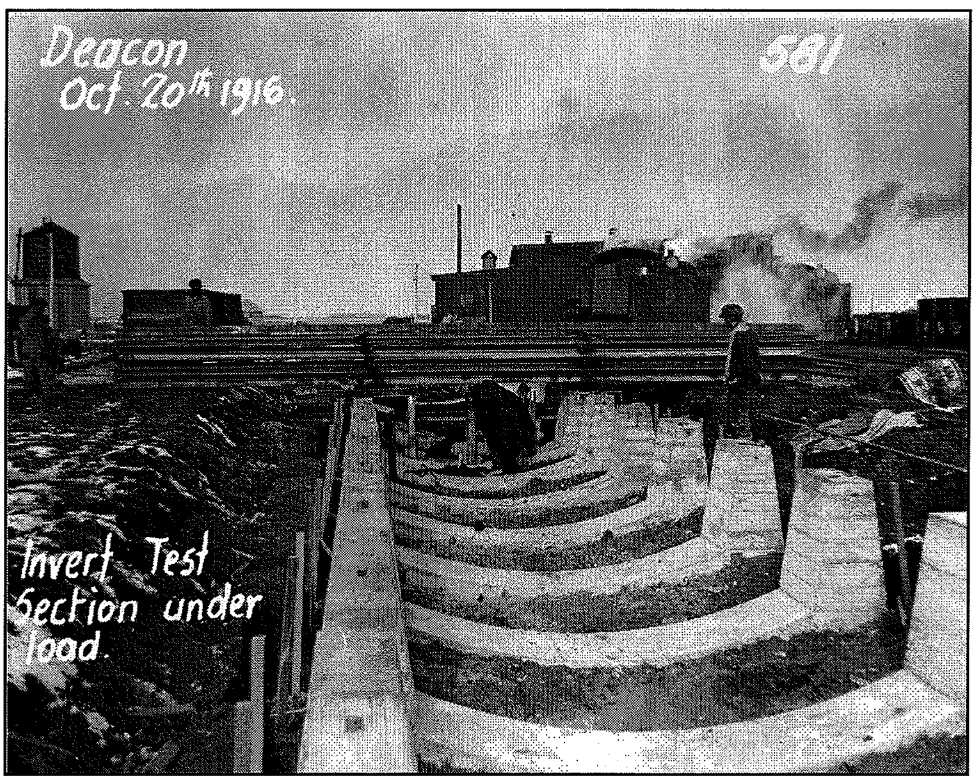


Figure 3-13. Invert testing program undertaken in 1916. (Construction Photograph No. 581)

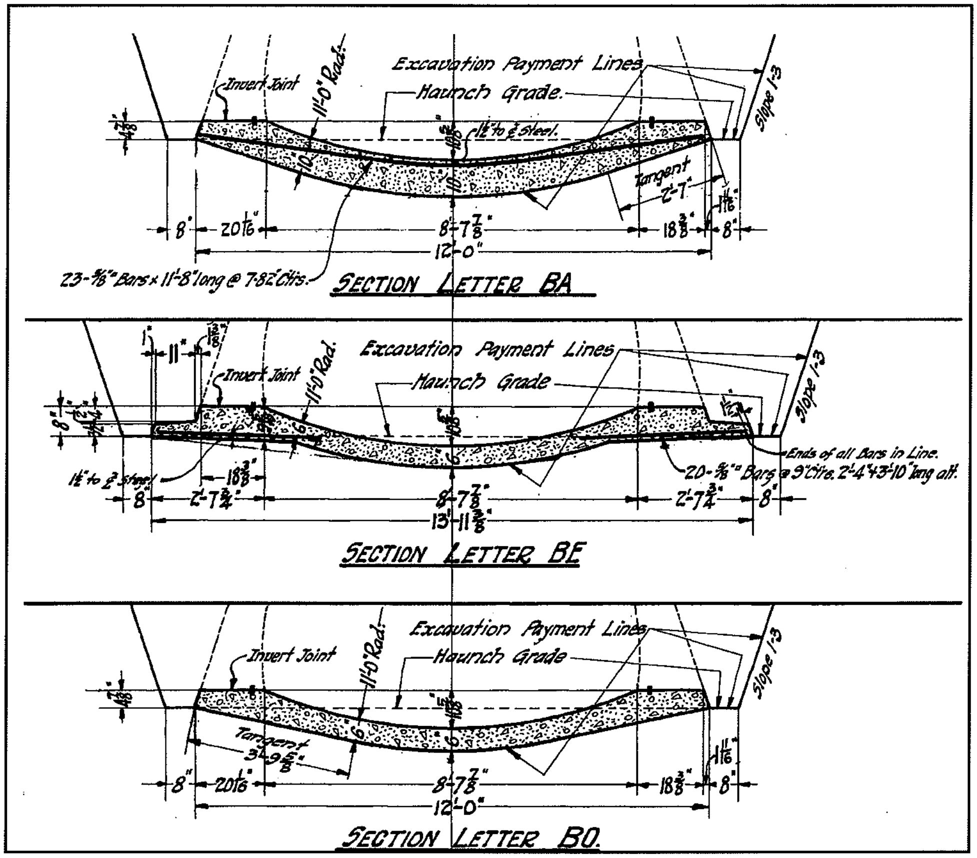


Figure 3-14. Three invert sections used for construction in 1917 and 1918 based on the results of the invert testing program from 1916. The inverts in this figure are for the 'B' aqueduct section. The same invert shapes were used for all the aqueduct section types constructed in 1917 and 1918. The 'A' invert section was used on what was considered to be the '*worst*' foundation conditions; the 'E' invert was used where the foundation conditions were considered to be '*better than the worst, but poorer than the best*'; and the 'O' invert section was used on what was considered to be the '*best*' foundation conditions. (from GWWD Drawing B-304)

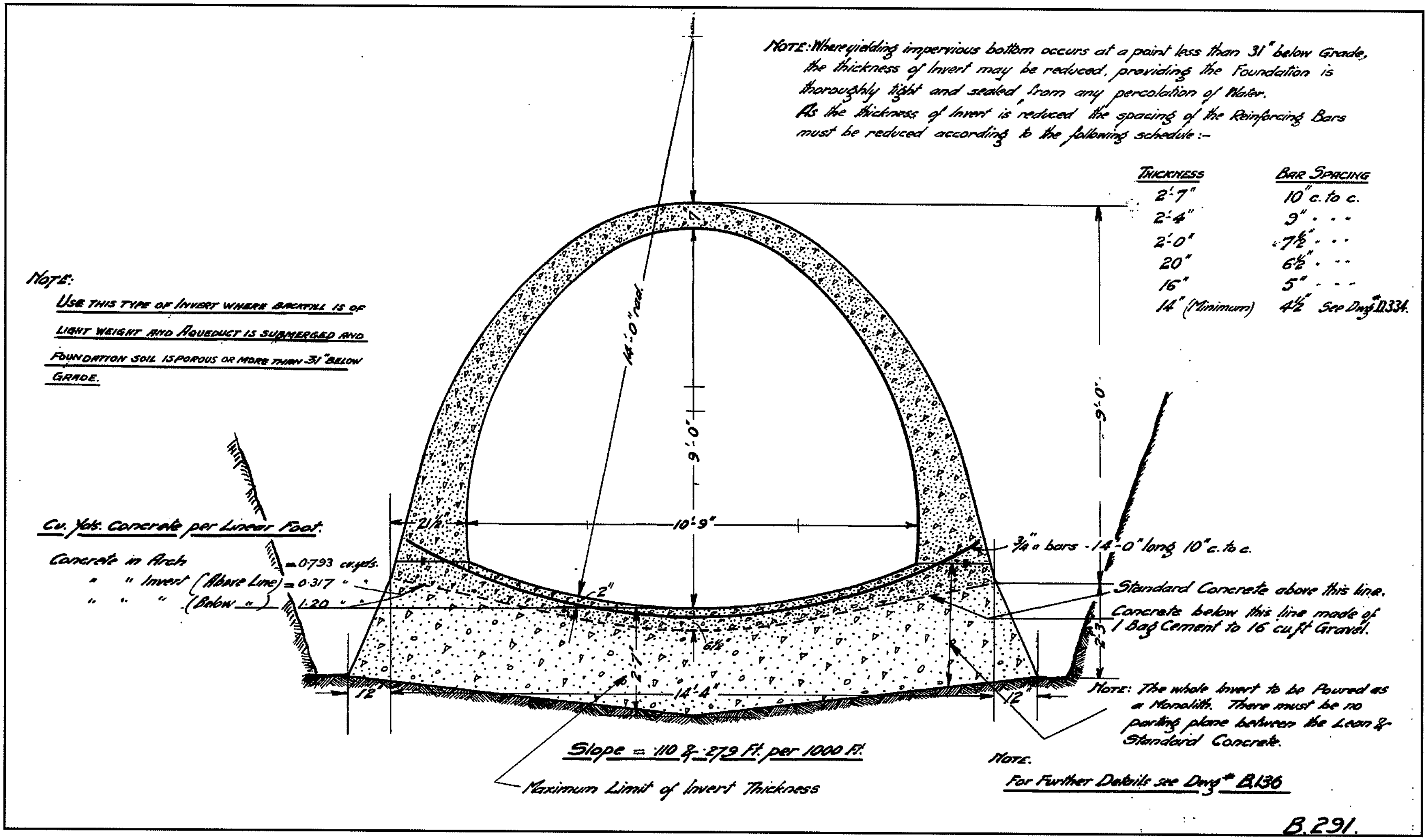


Figure 3-15. Special gravity invert used to prevent buoyancy. The thickness of the invert was sometimes reduced if a foundation soil of low permeability was encountered before the full depth of the invert was excavated. (from GWWD Drawing B-291)

Figure 3-16. Cast-in-place inverts constructed in 4.57 metres lengths. (Construction Photograph No. 418)

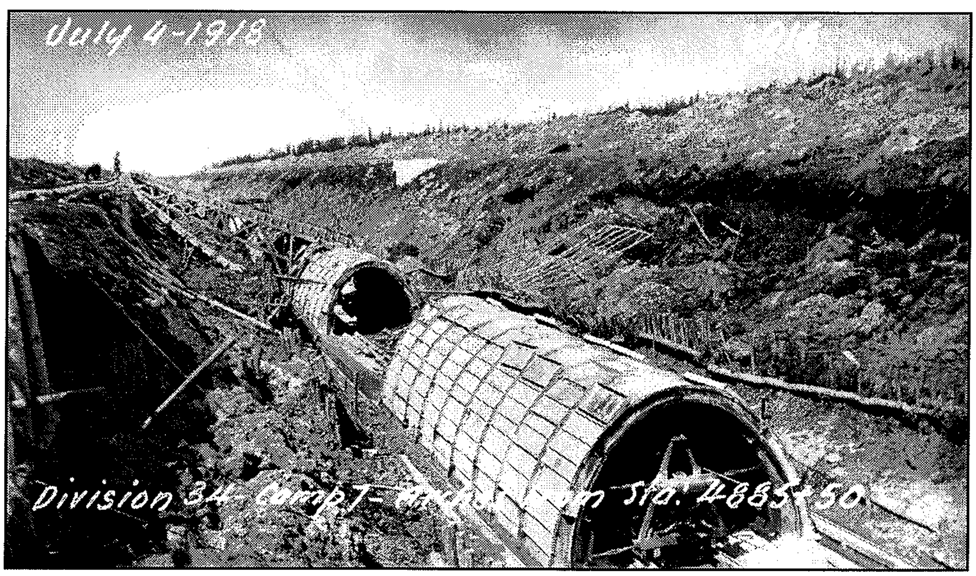


Figure 3-17. Arches cast-in-place using steel forms. The arch segments were cast in 13.7 metres lengths that covered three, 4.57 metres invert segments.
(Construction Photograph No. #916)

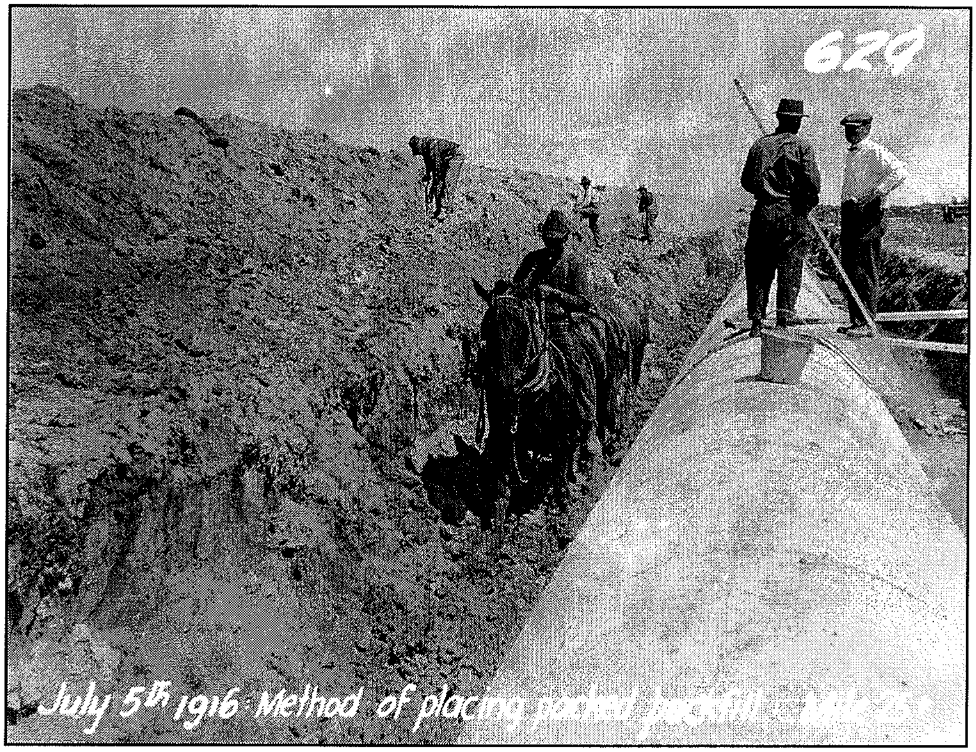


Figure 3-18. Compacting backfill beside the lower extremity of the arch leg. (Construction Photograph No. 629)

Figure 3-19. Steep trench side slopes (3V:1H) were excavated where possible. Side slopes were flattened to 1V:1H when loose or soft soils were encountered.
(Construction Photograph No. 919)

Figure 3-20. Approximately 1.0 to 1.5 metres of base heave in the aqueduct trench, Mile 85.5. Note the spoil banks adjacent to the excavation. (Construction Photograph No. 334)

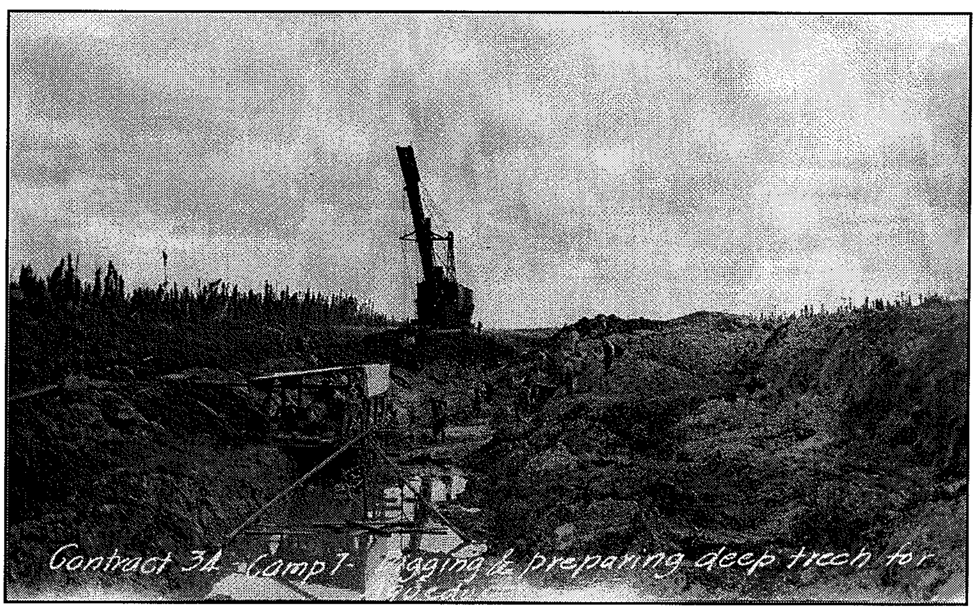


Figure 3-21. Slope failure of the aqueduct trench near Mile 90. (Construction Photograph No. 360)

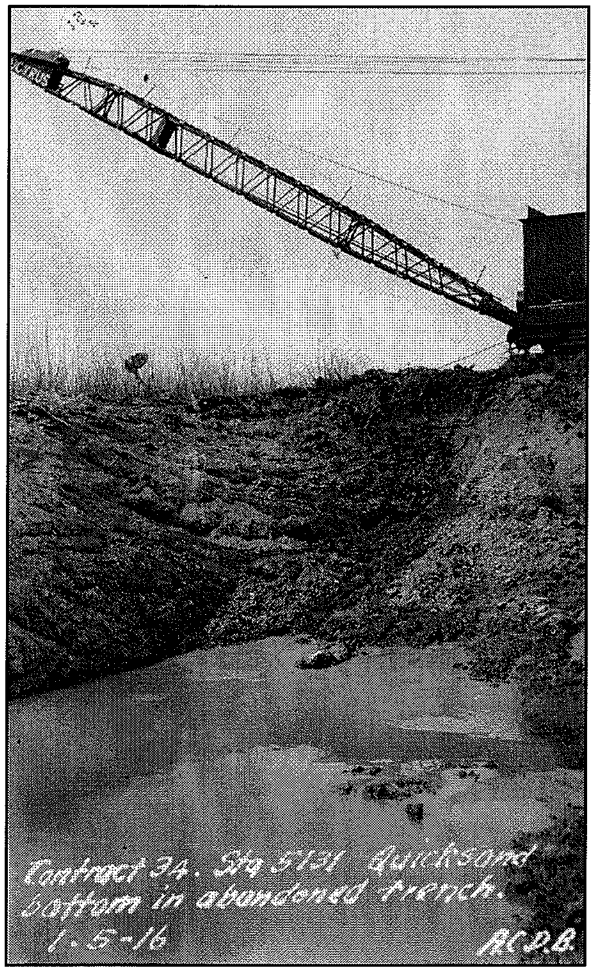


Figure 3-22. Groundwater piping into the base of the aqueduct trench at Mile 97.2. (Construction Photograph No. 373)

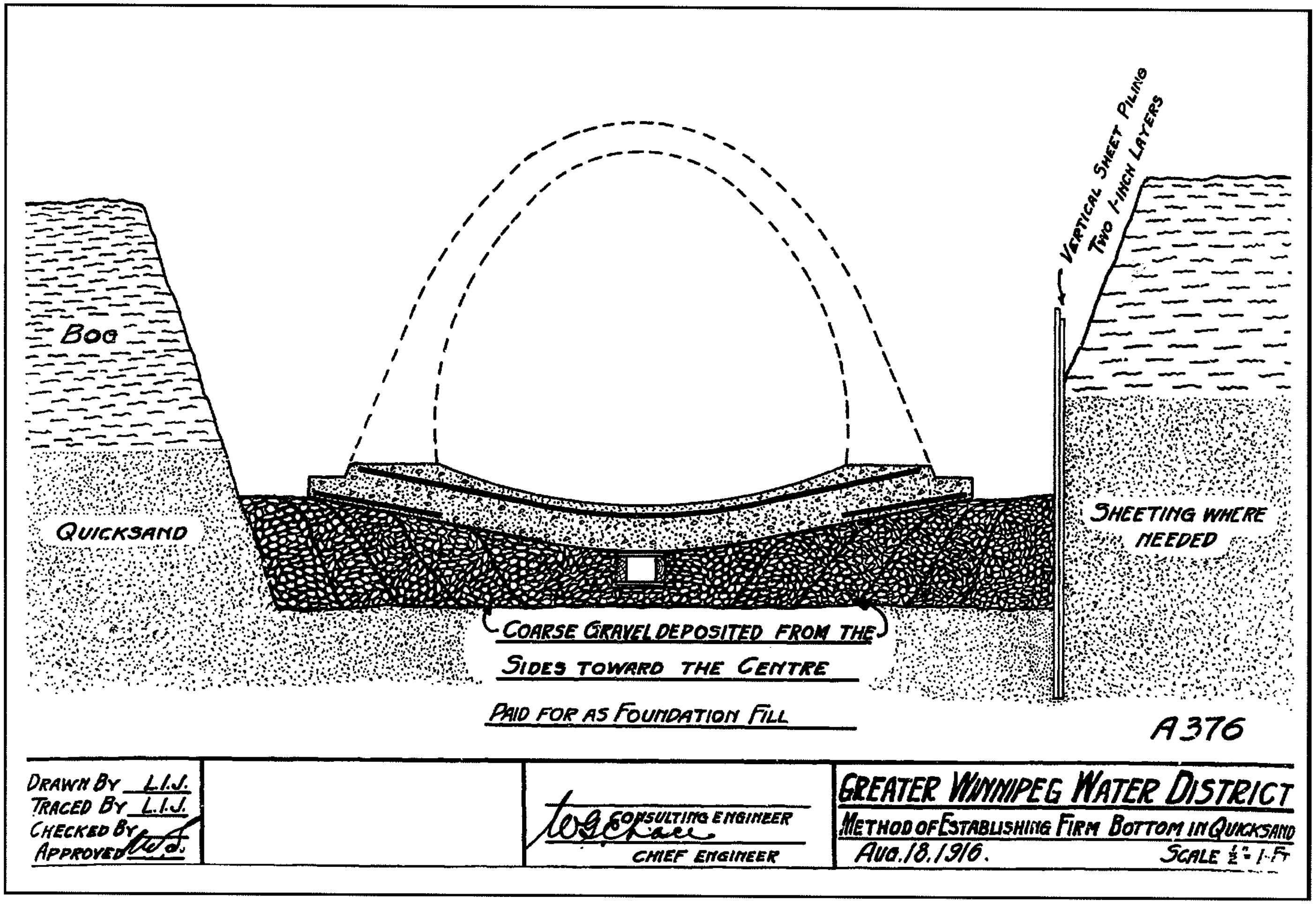


Figure 3-23. Method used to establish a '*firm*' foundation when groundwater piping conditions were encountered. (from GWWD Drawing A-376)

Figure 3-24. Wooden box drains installed just below grade to help control groundwater seepage. (Construction Photograph No. 442)

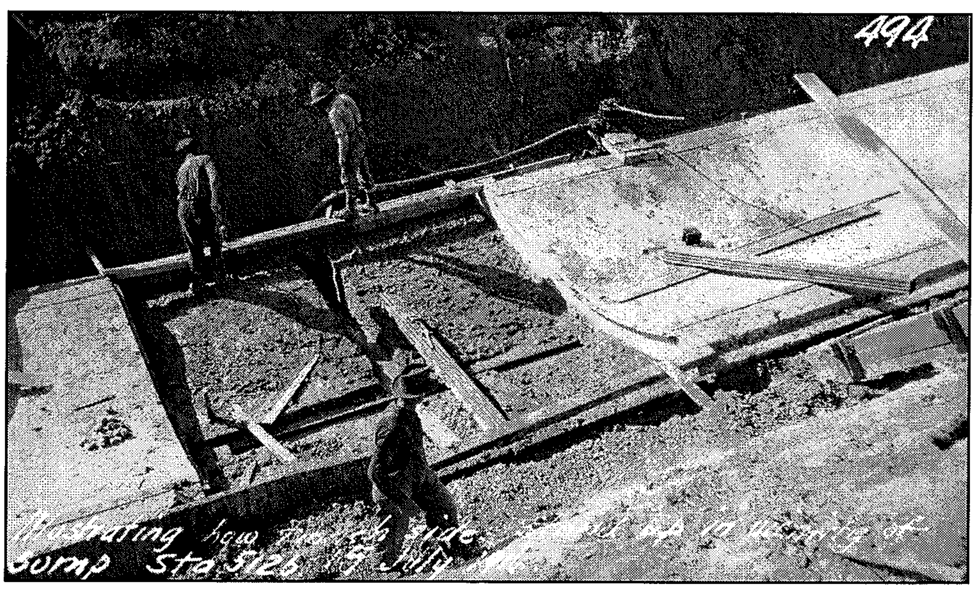


Figure 3-25. Box drain and offtake to drainage sump. (Construction Photograph No. 494)

Figure 3-26. Slope failure of the aqueduct trench near Mile 92.5. (Construction Photograph No. 920)

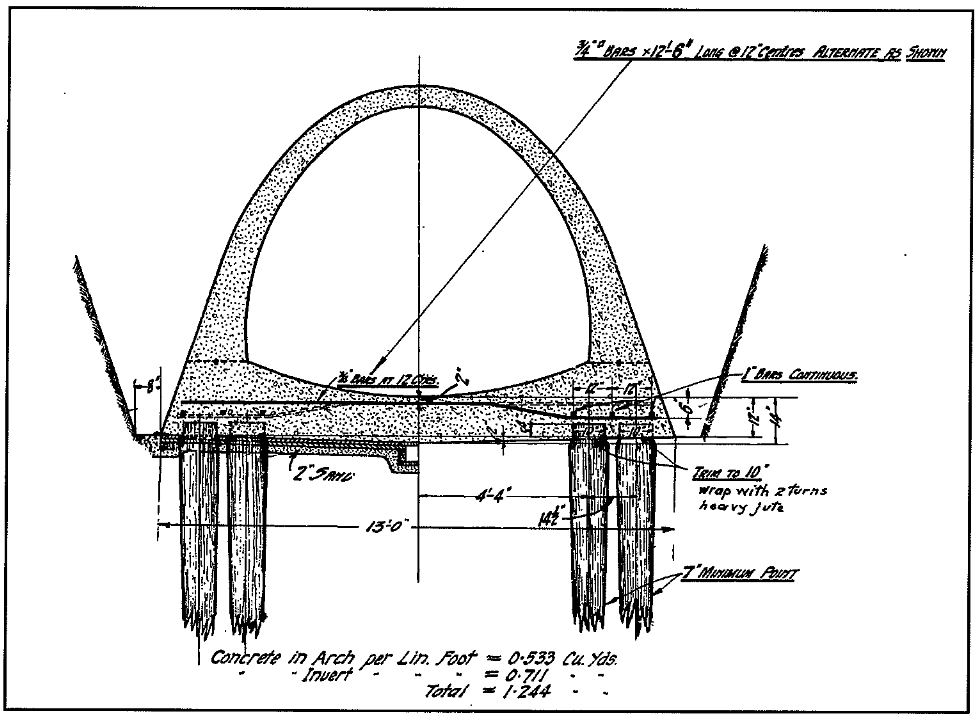


Figure 3-27. Timber pile foundation used where 'flowing clays' were encountered. (from GWWD Drawing B253)

Figure 3-28. Sand and gravel backfill used where the trench was over-excavated to remove organic soils. (Construction Photograph No. 190)

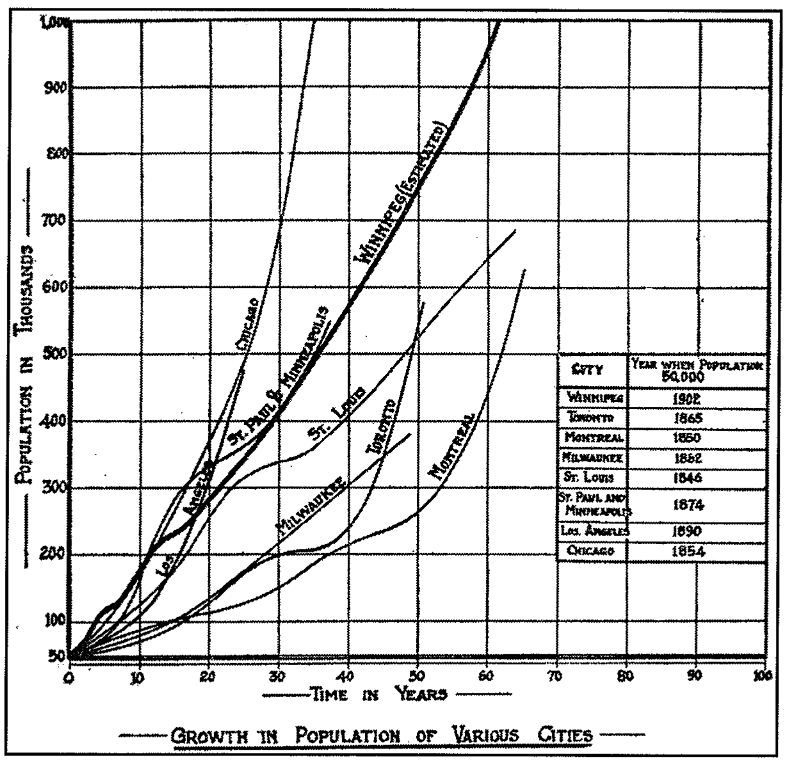


Figure 3-29. Predicted population growth for Winnipeg in the early 1900's. (from GWWD Drawing A-700)

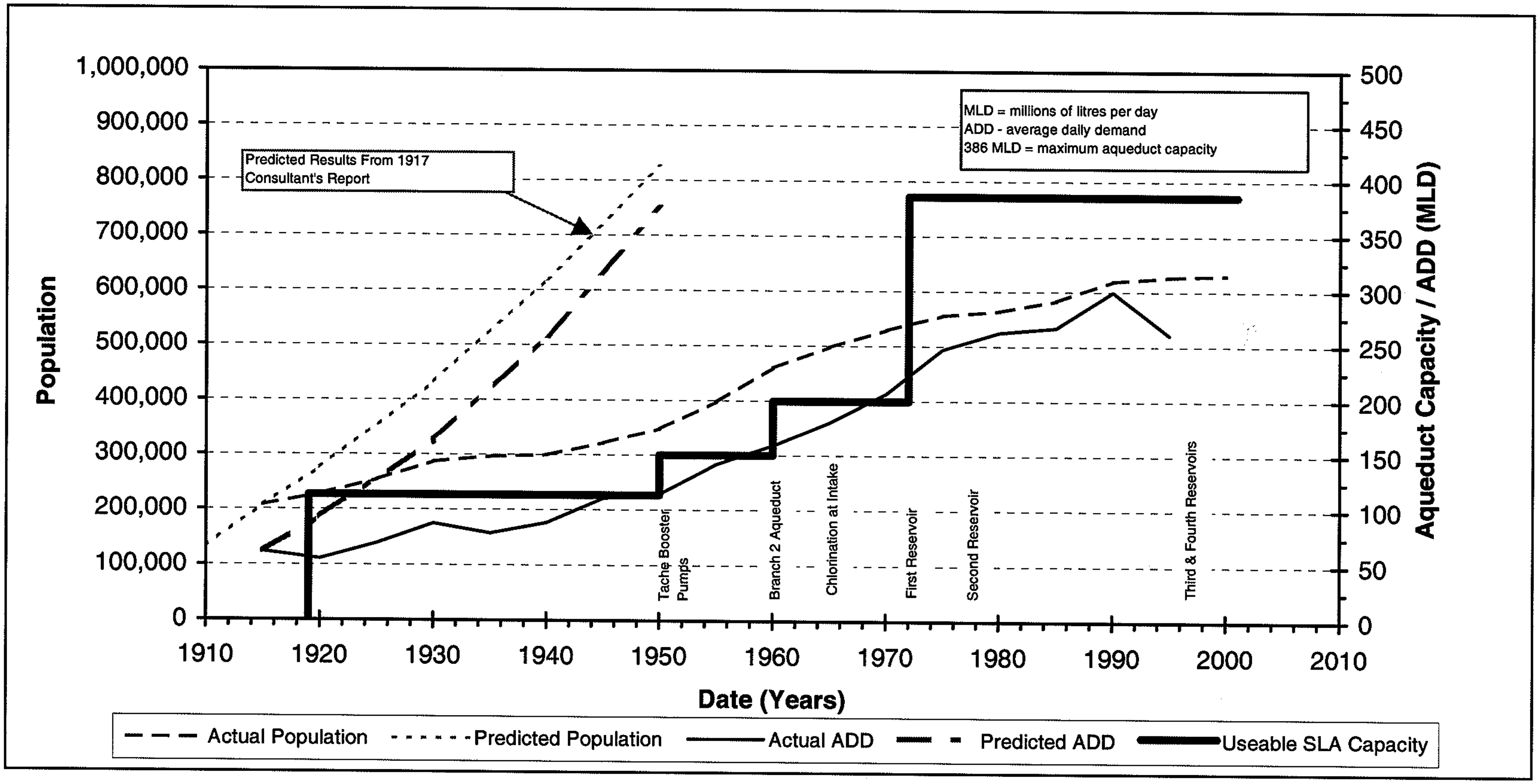


Figure 3-30. Predicted versus actual population and average daily water demand (ADD). The aqueduct capacity was developed in stages as the average daily water demand increased.

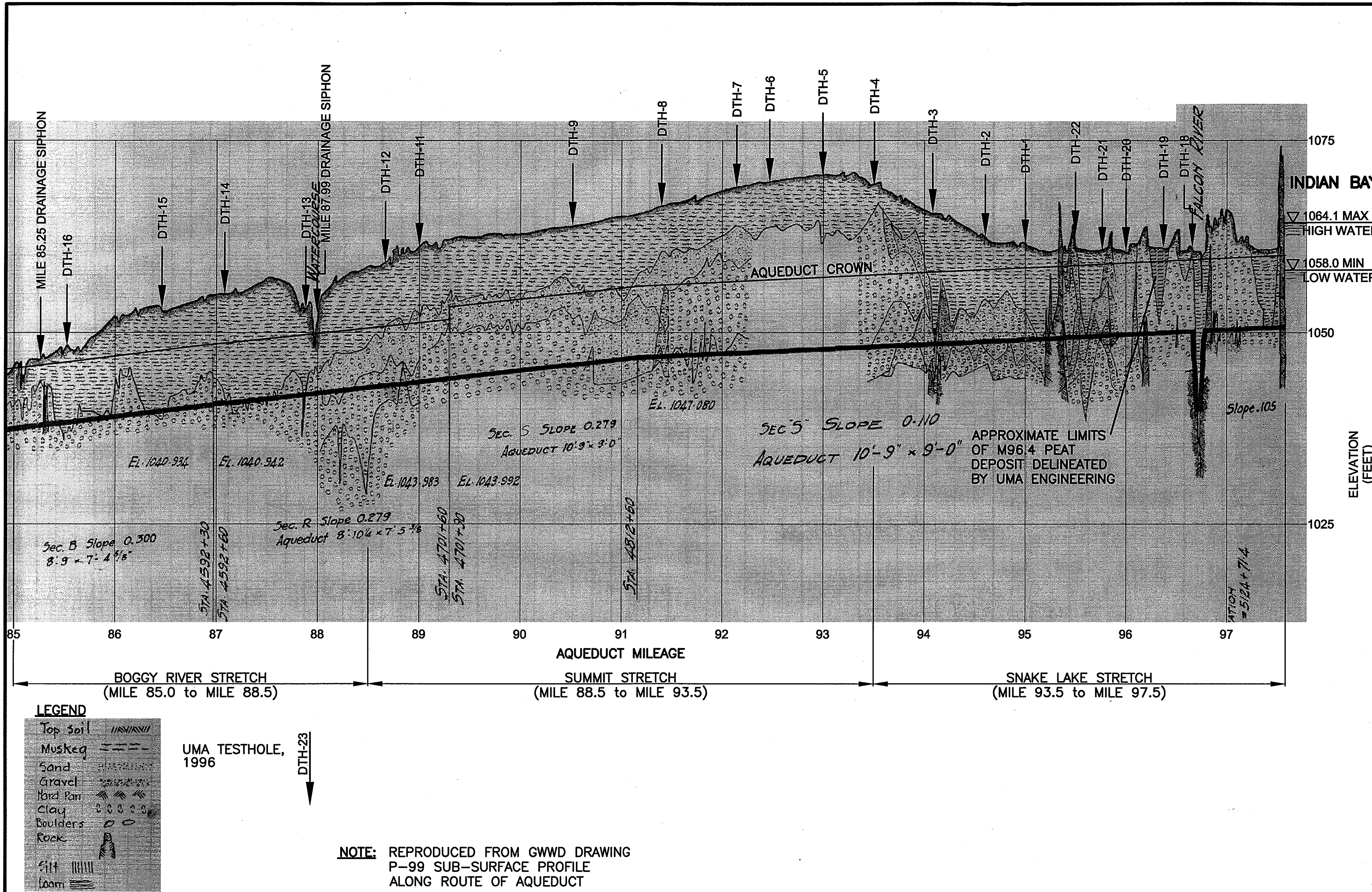


Figure 4-1. Sub-surface profile and detailed test hole locations.

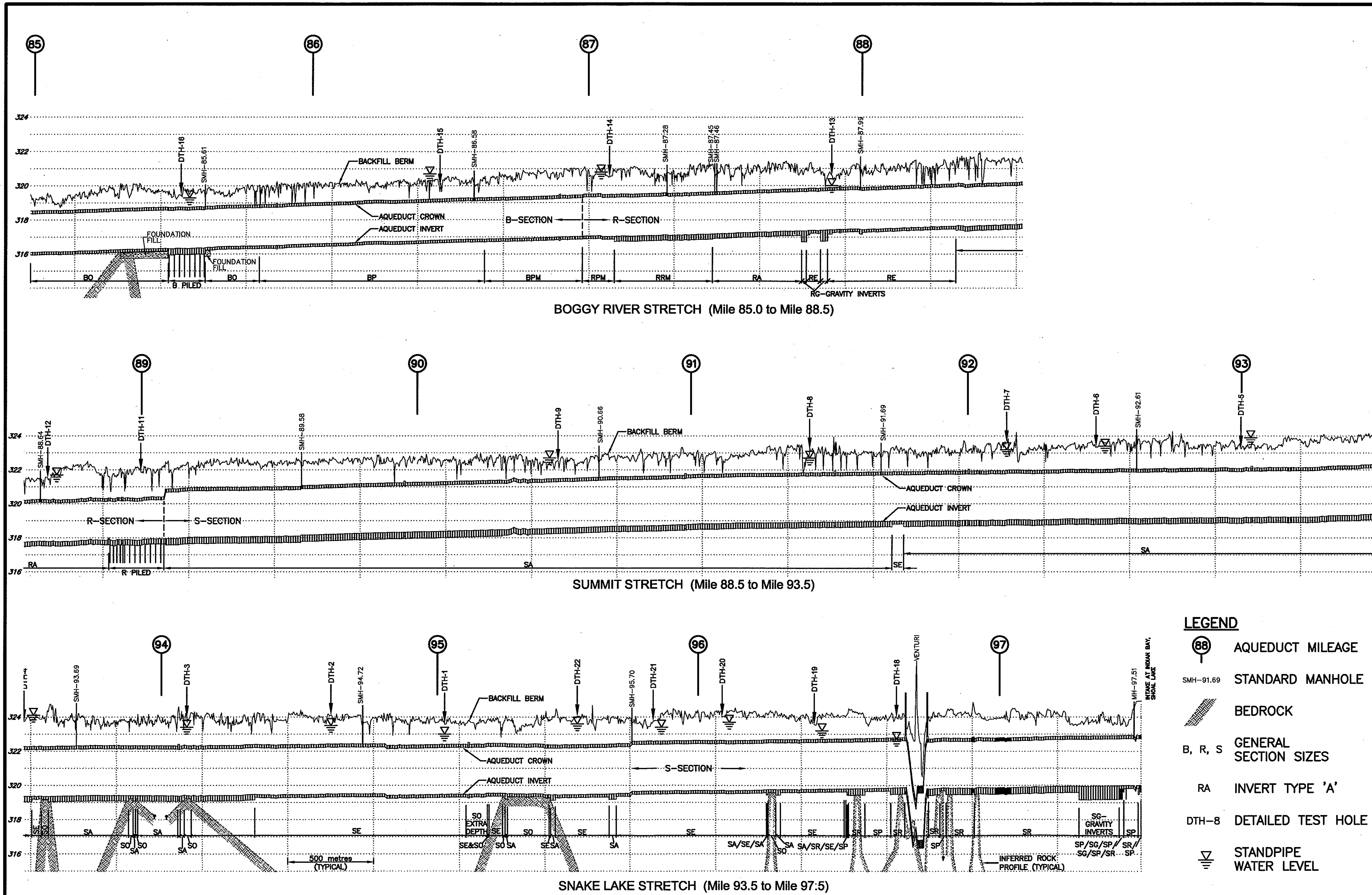


Figure 4-2. Aqueduct profile and detailed test hole locations.

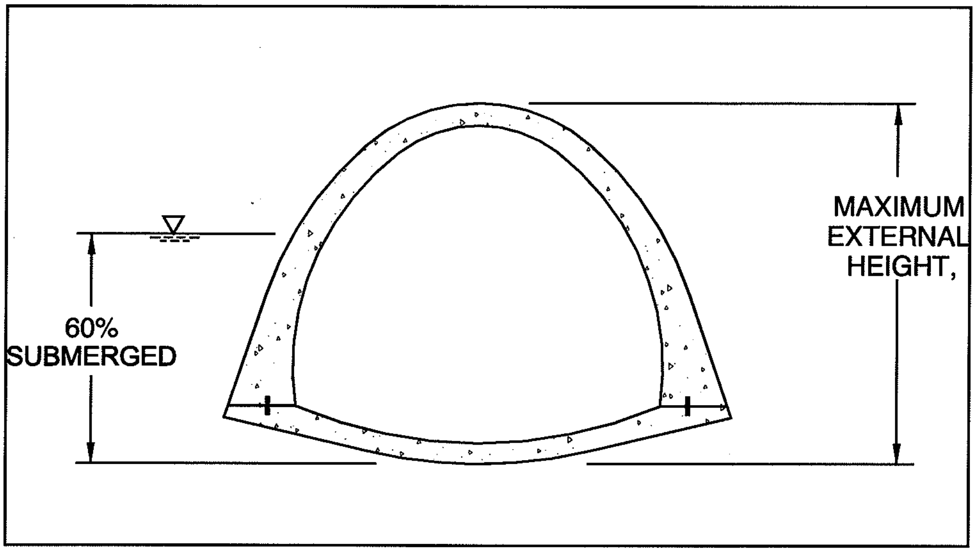


Figure 4-3. Minimum degree of aqueduct submergence required for buoyancy to occur is about 60 percent of the maximum external height.

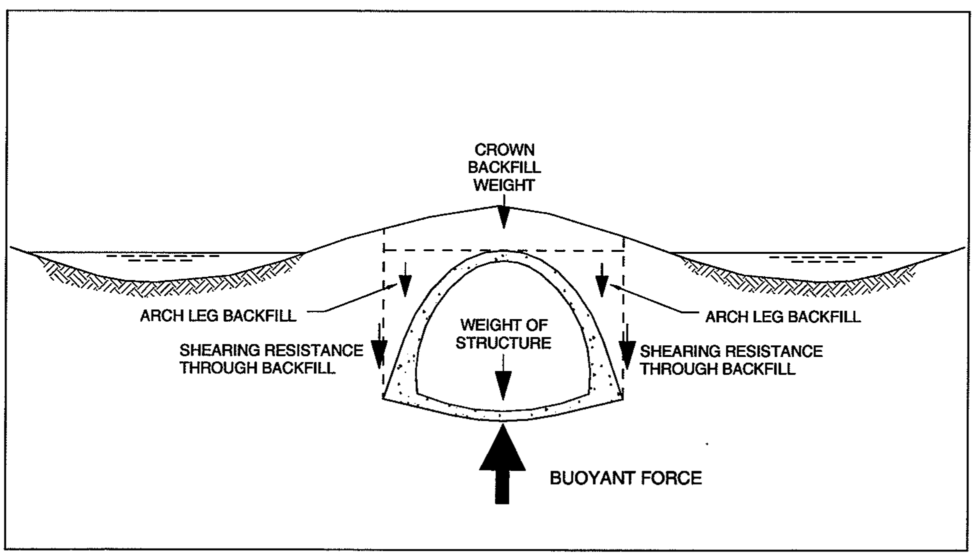


Figure 4-4. Simplified force diagram showing the resisting forces and the net upward hydrostatic or buoyant force.

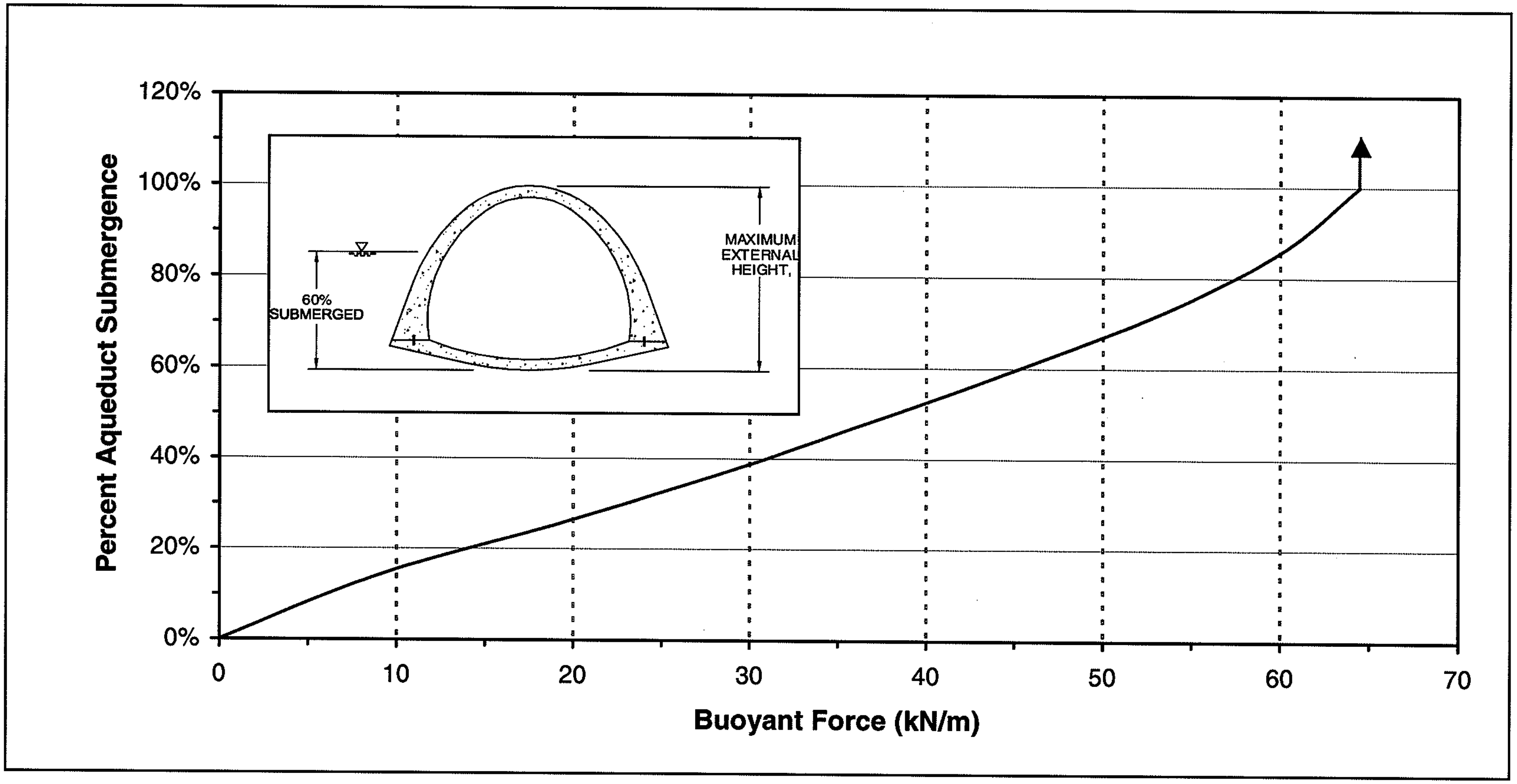


Figure 4-5. The buoyant force acting on the aqueduct changes with increasing degrees of submergence. The buoyant force is approximately equal to the weight of the aqueduct when it is about 60 percent submerged. The maximum buoyant force is generated when the water level is equal to or higher than the top of the aqueduct.

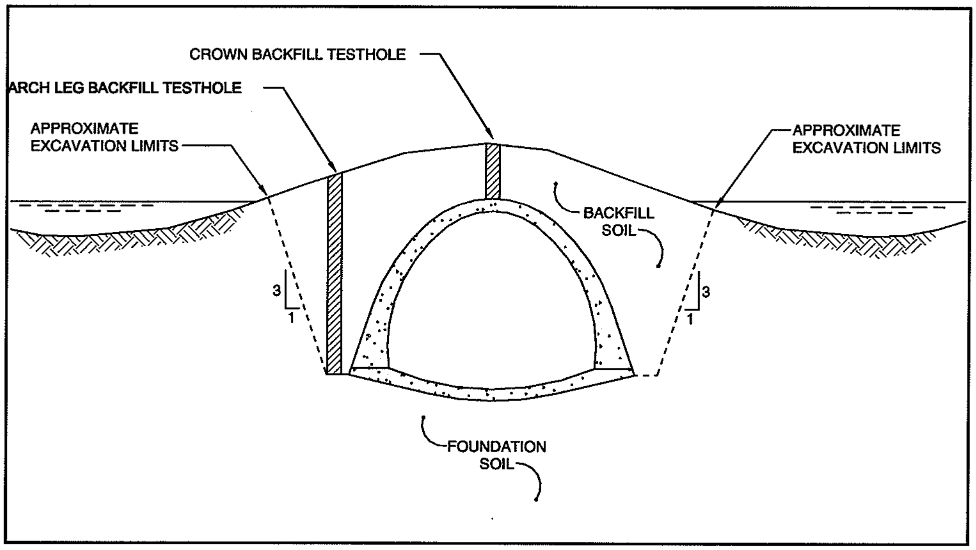


Figure 4-6. Typical locations of the backfill test holes.

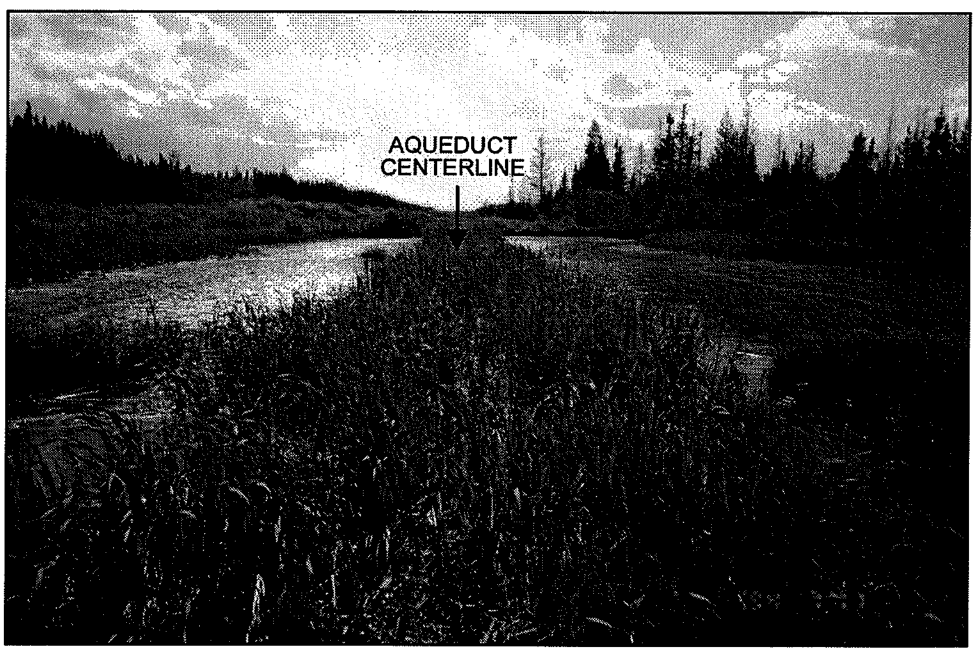


Figure 4-7. Typical site conditions in the Boggy River Stretch (Mile 85.0 to Mile 88.5).

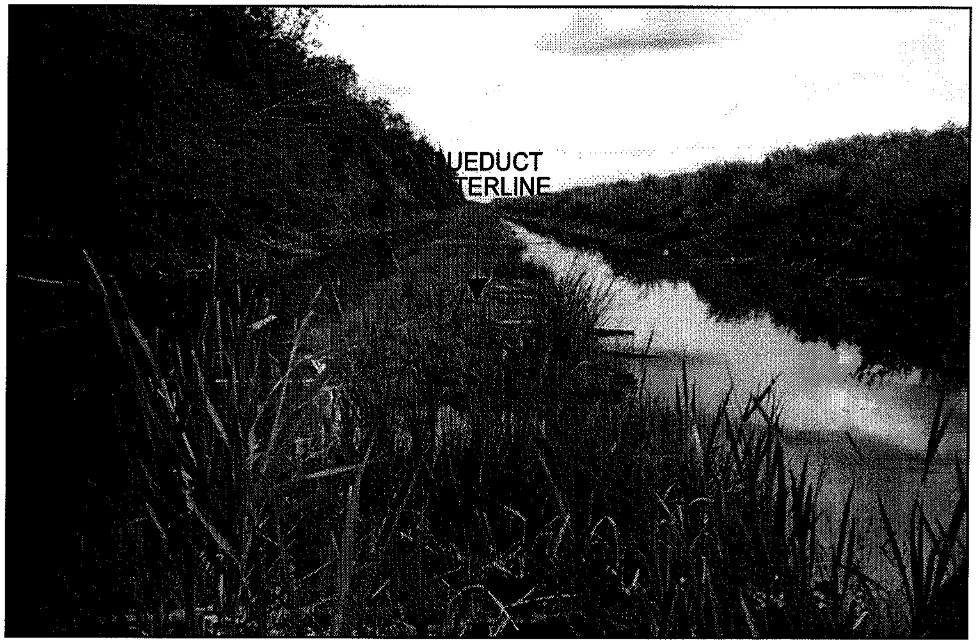


Figure 4-8. Typical site conditions in the Summit Stretch (Mile 88.5 to Mile 93.5). The higher ground levels on either side of the ditches are a result of not backfilling the original excavation to natural grade.

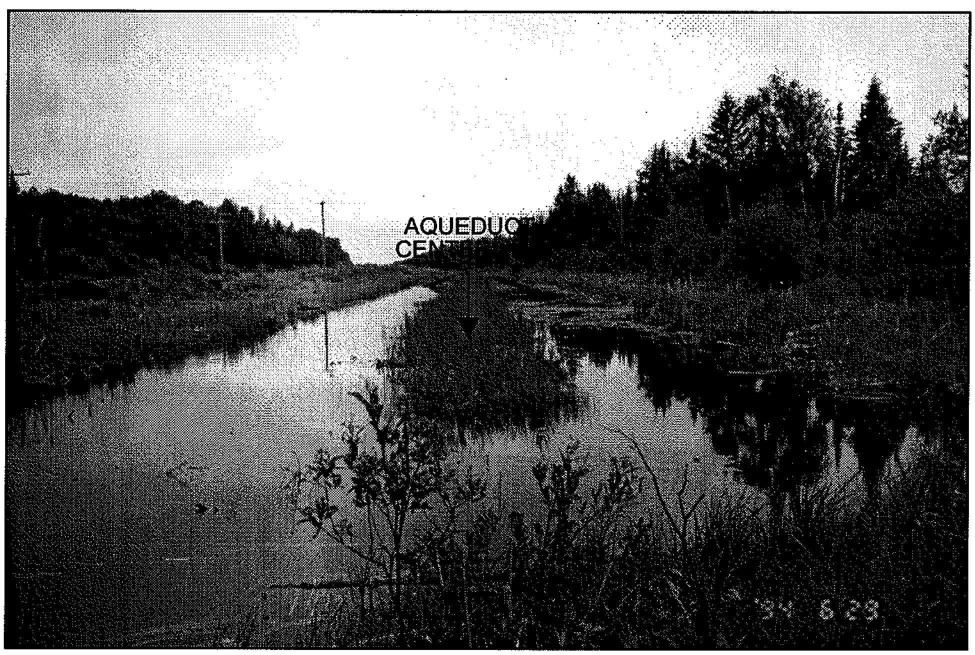


Figure 4-9. Typical site conditions in the Snake Lake Stretch (Mile 93.5 to Mile 97.5).

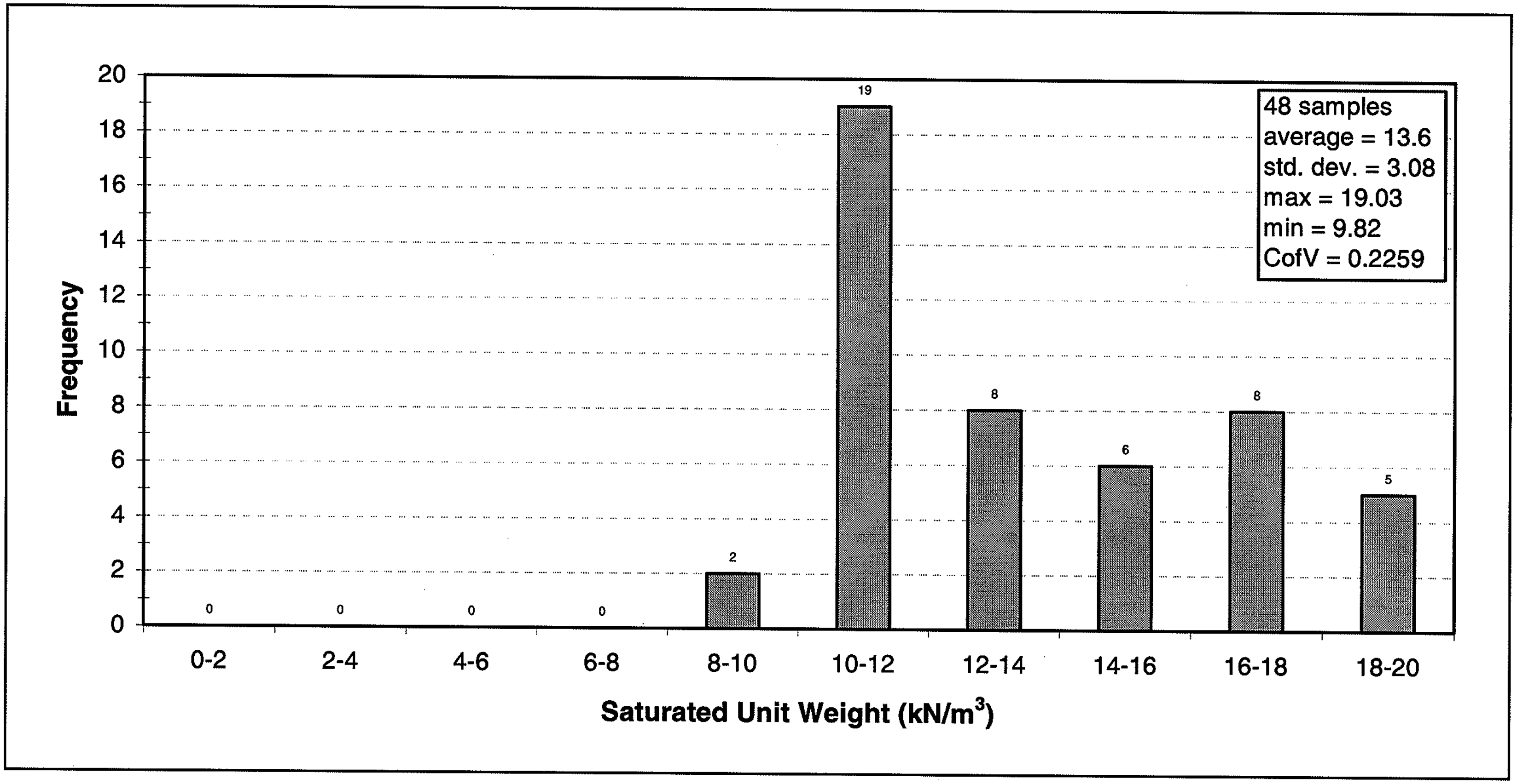


Figure 4-10. Histogram of saturated backfill unit weights measured between Mile 85.0 and Mile 95.0. These measurements were used to develop the relationship between dry unit weight and water content shown in Figure 4-11.

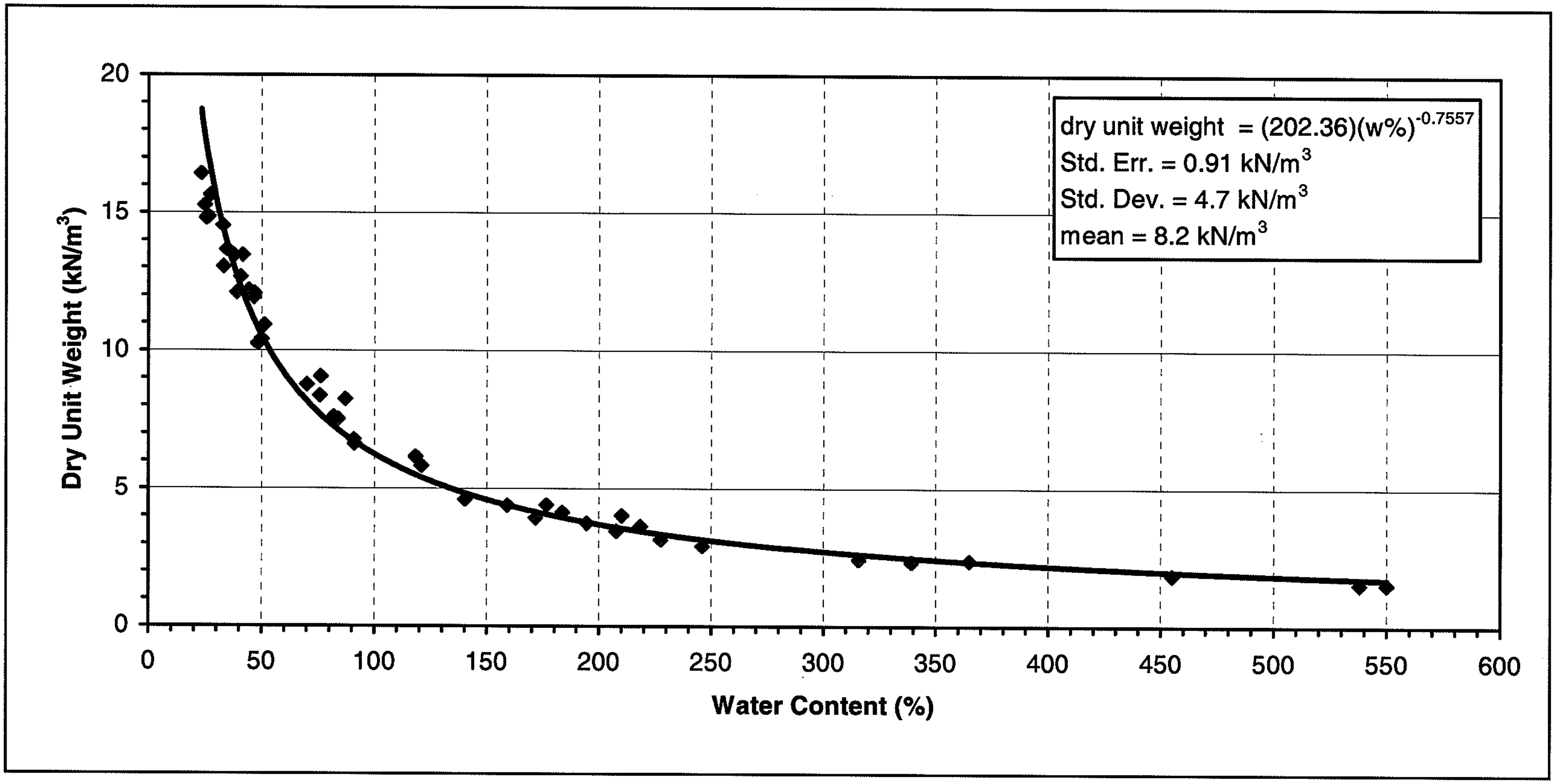


Figure 4-11. Relationship between dry backfill unit weight and water content determined using non-linear regression analysis.

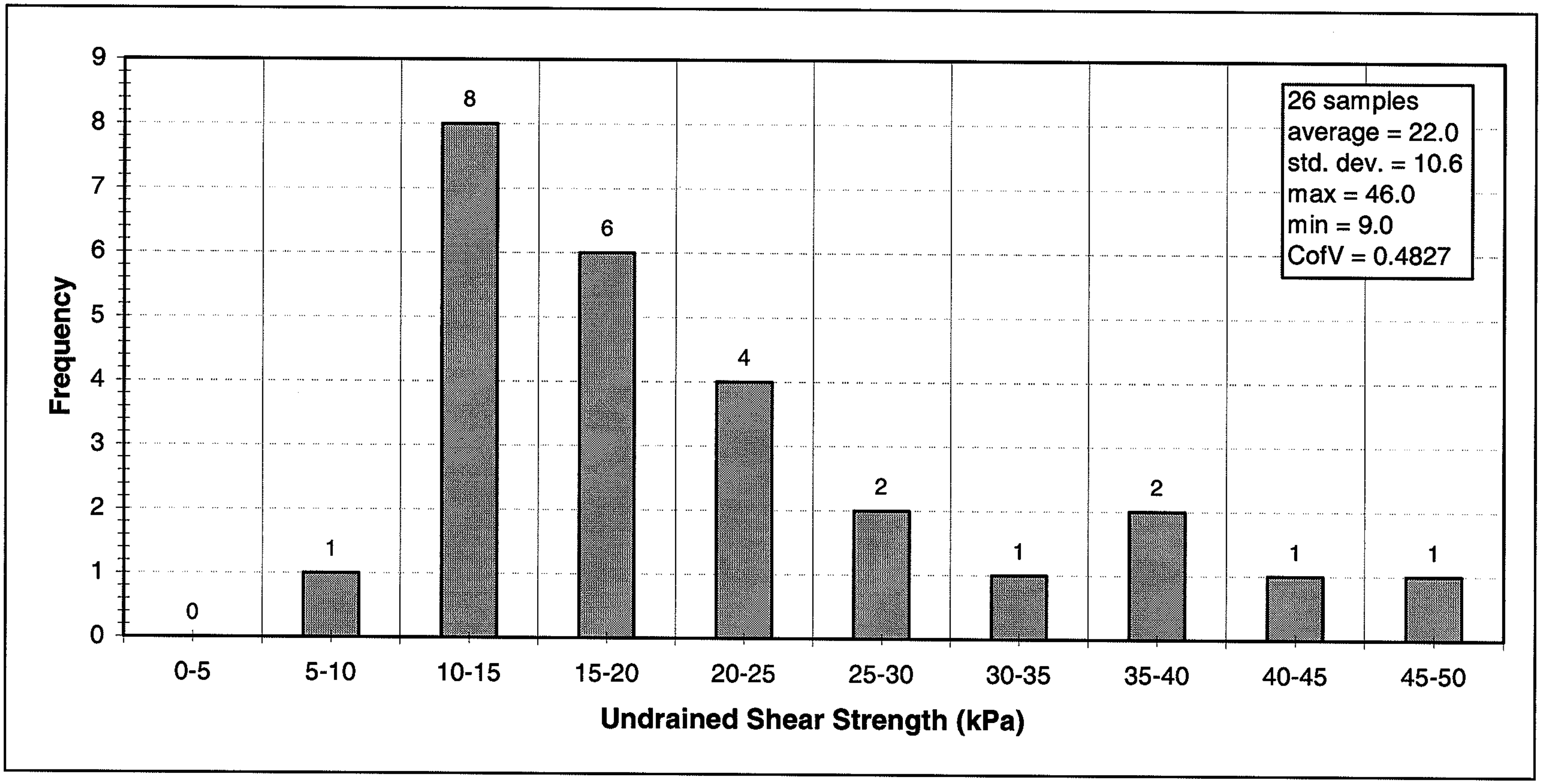


Figure 4-12. Histogram of shear strengths measured in the backfill soils adjacent to the aqueduct.

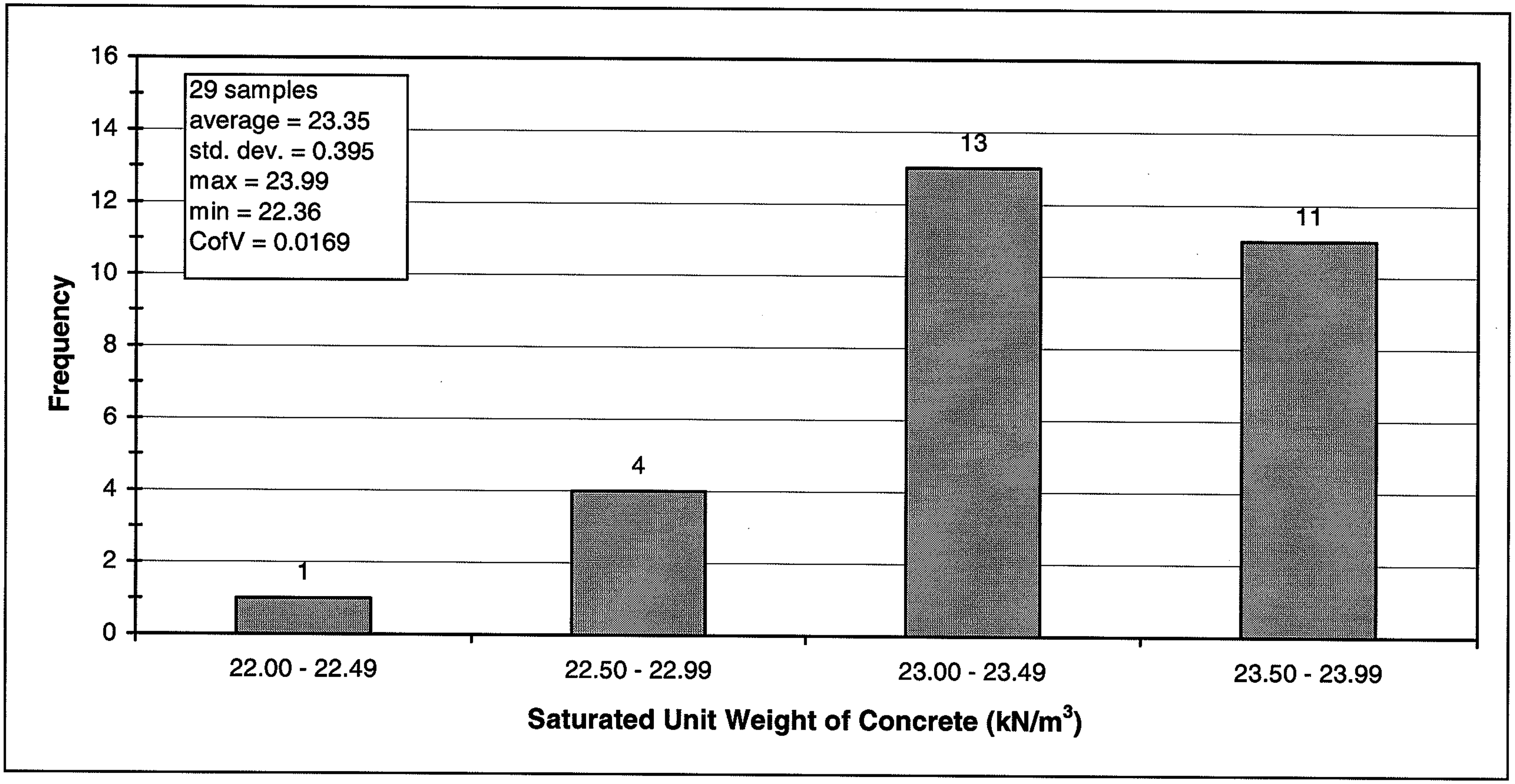


Figure 4-13. Histogram of saturated concrete unit weights measured from the concrete cores taken between Mile 85.6 and Mile 95.7.

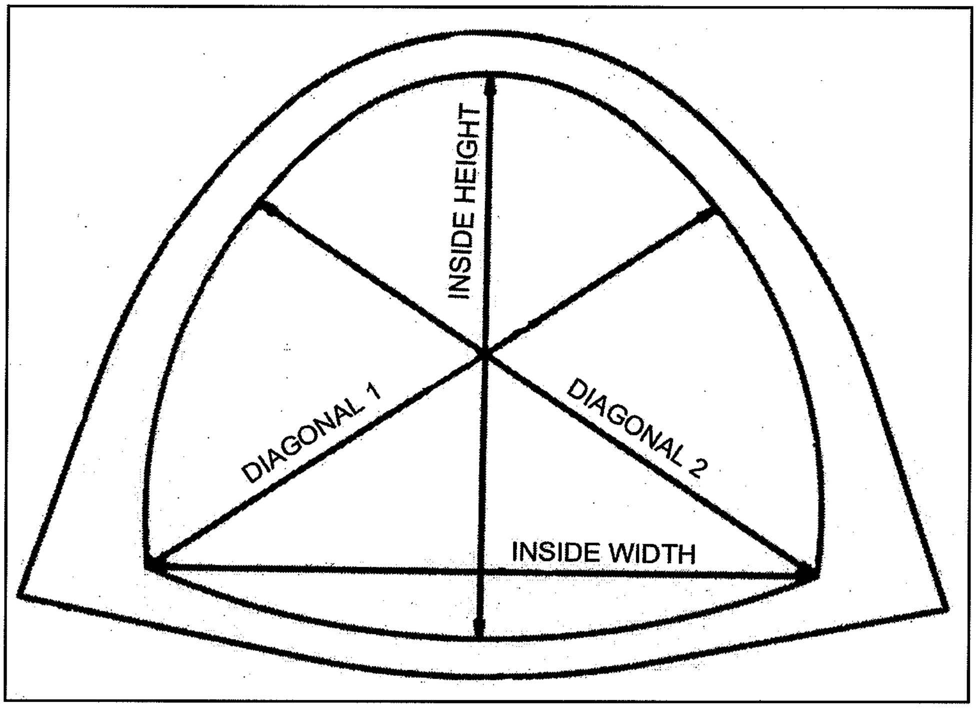


Figure 4-14. Diagram of the locations where internal measurements used in the buoyancy analysis work were taken.

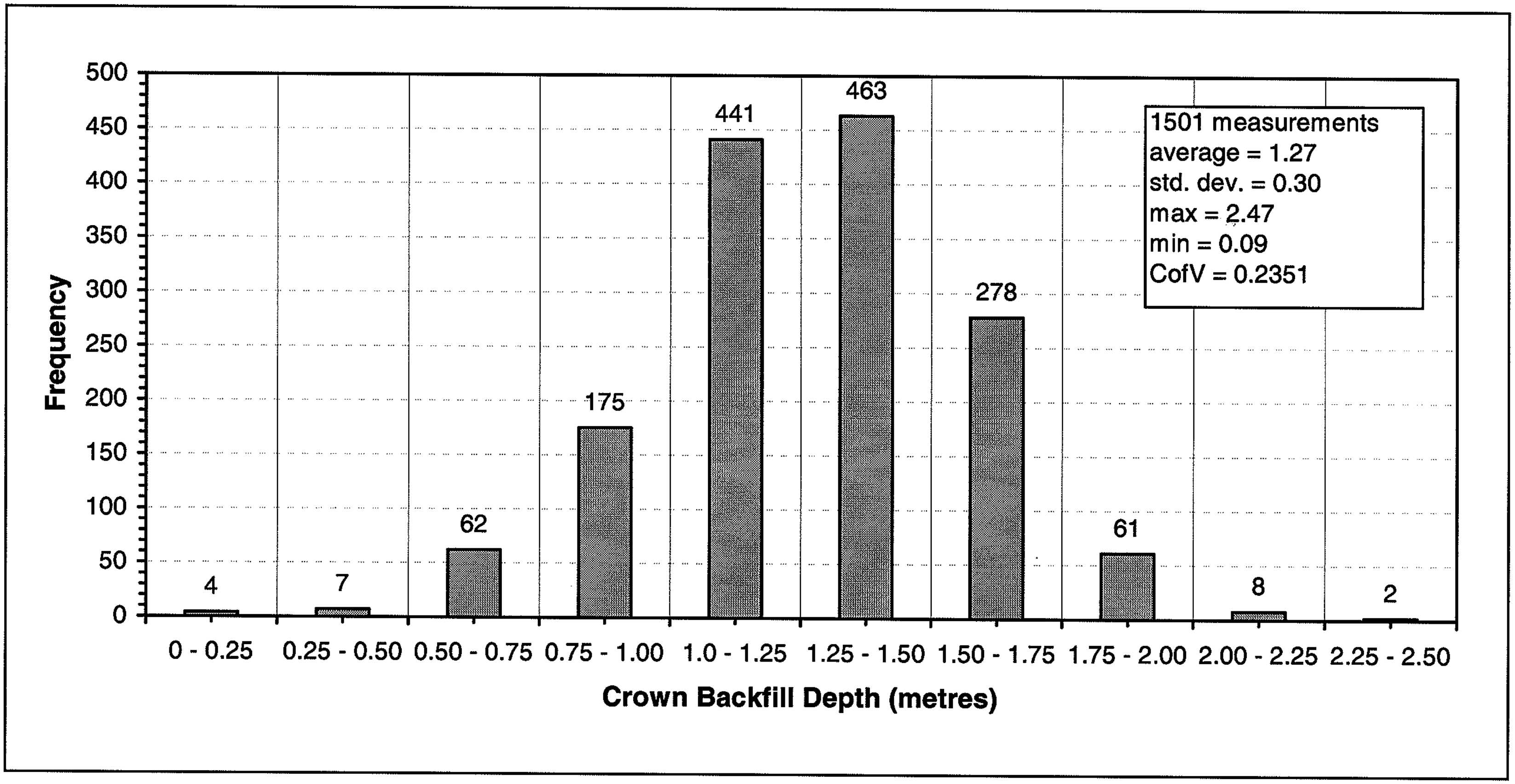


Figure 4-15. Histogram of the crown backfill depths determined from the profile survey of the backfill berm.

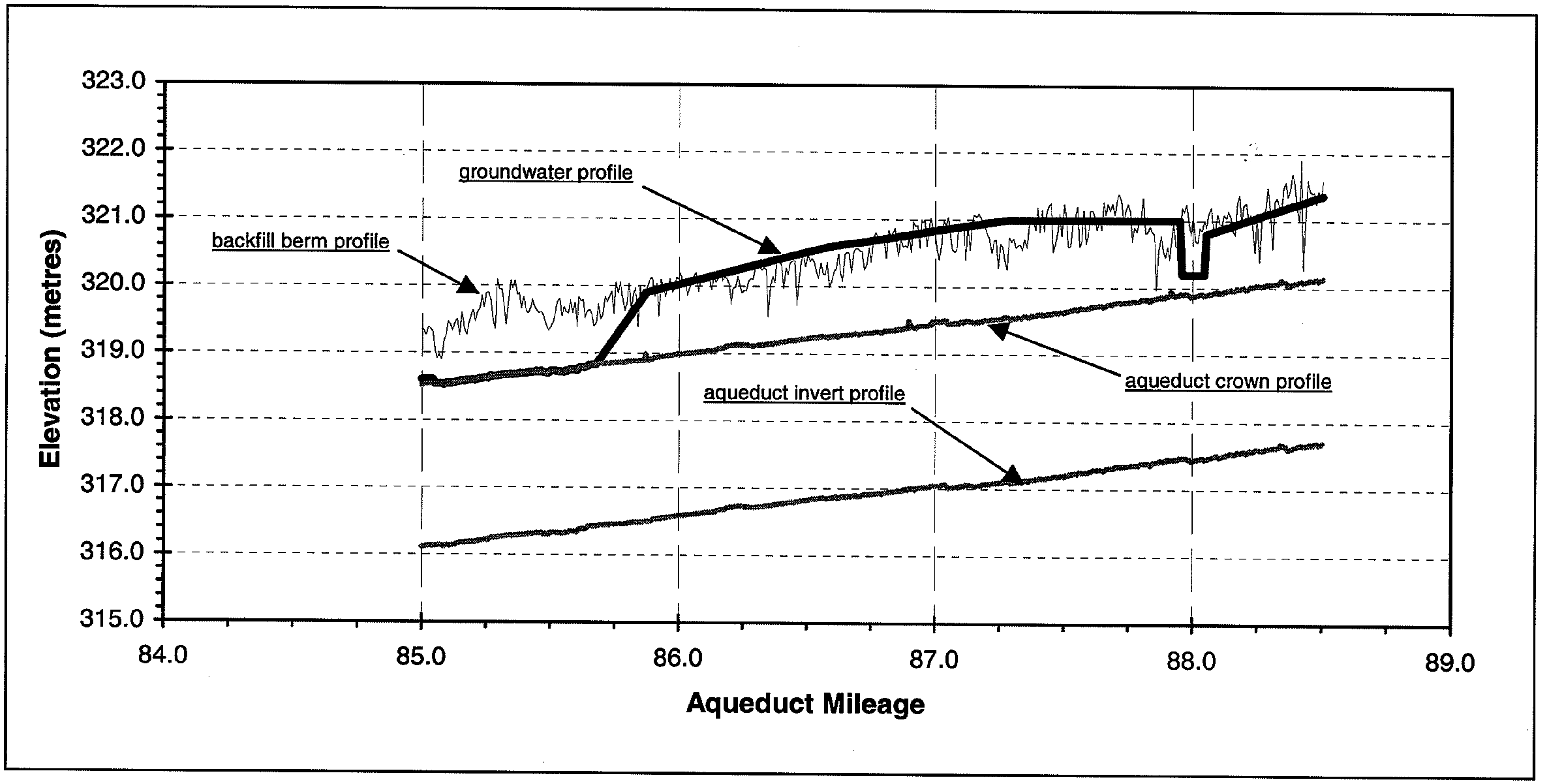


Figure 4-16. Groundwater profile delineated from survey data taken in the Boggy River Stretch (Mile 85.0 to Mile 88.5).

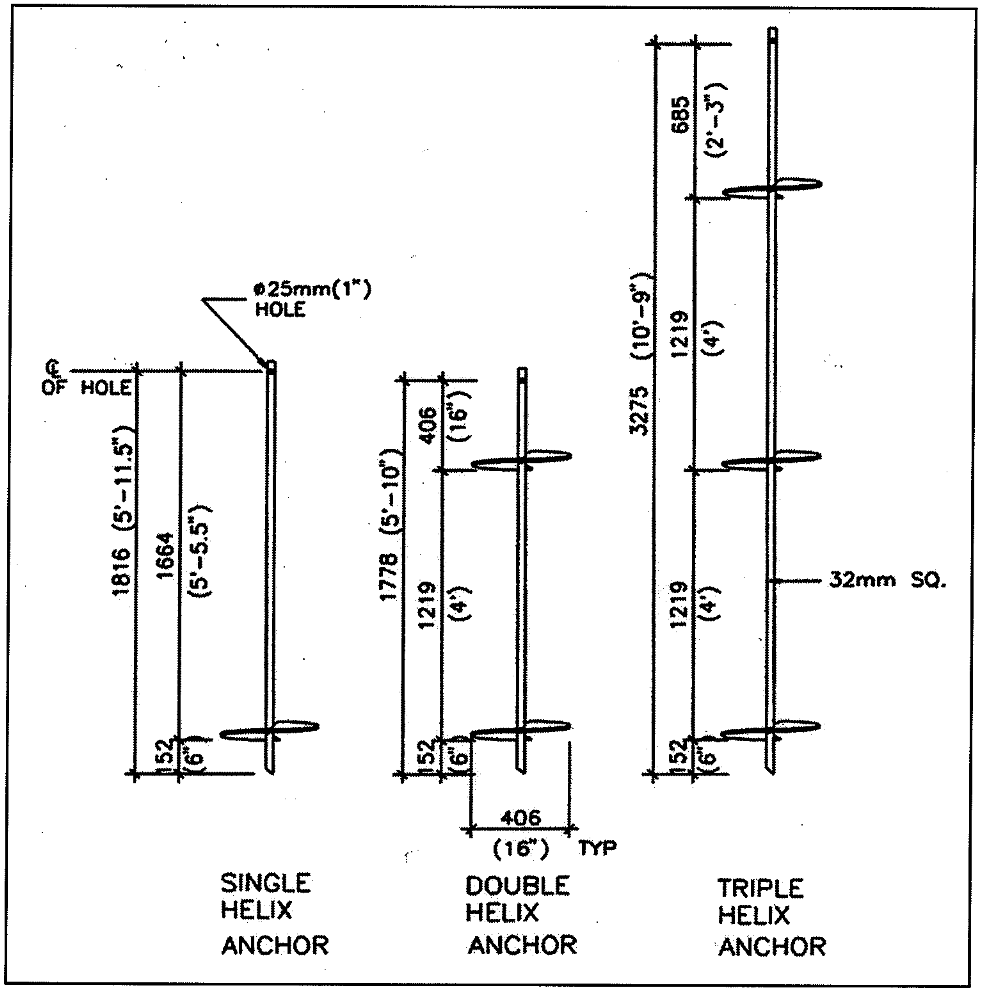


Figure 4-17. Details of the helical screw type anchors used in the soil anchor testing program.

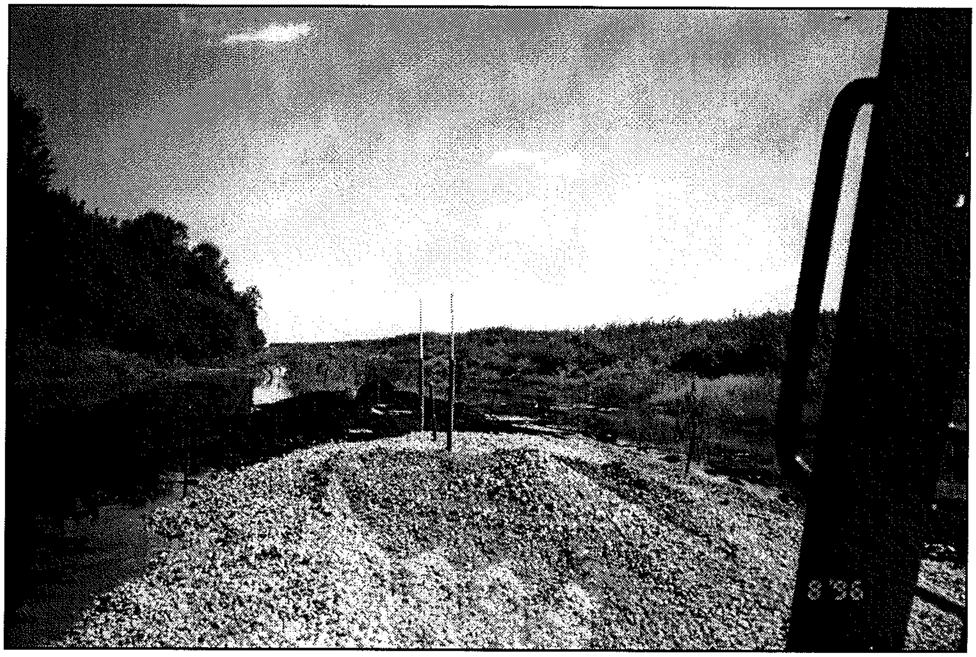


Figure 4-18. Typical site conditions at the trial ballasting site. Note the geotextile in the background used as a separation layer between the original backfill soils and the granular ballast.

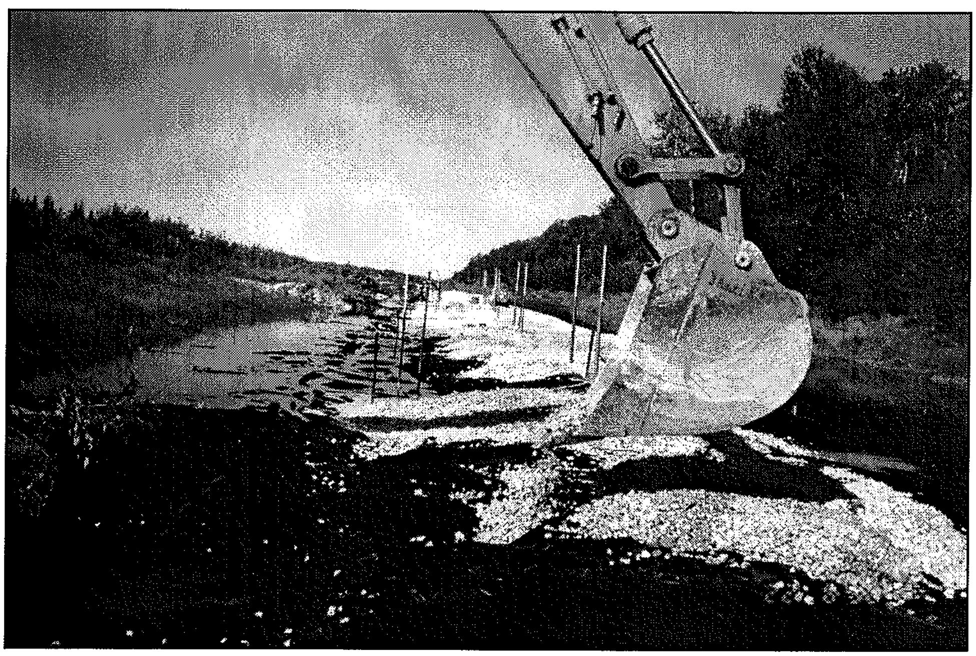


Figure 4-19. Placing granular ballast on the aqueduct at the trial ballasting site. Note the monitoring instrumentation located along the centreline of the aqueduct and the instrumentation offset from centreline (Section 4.10).

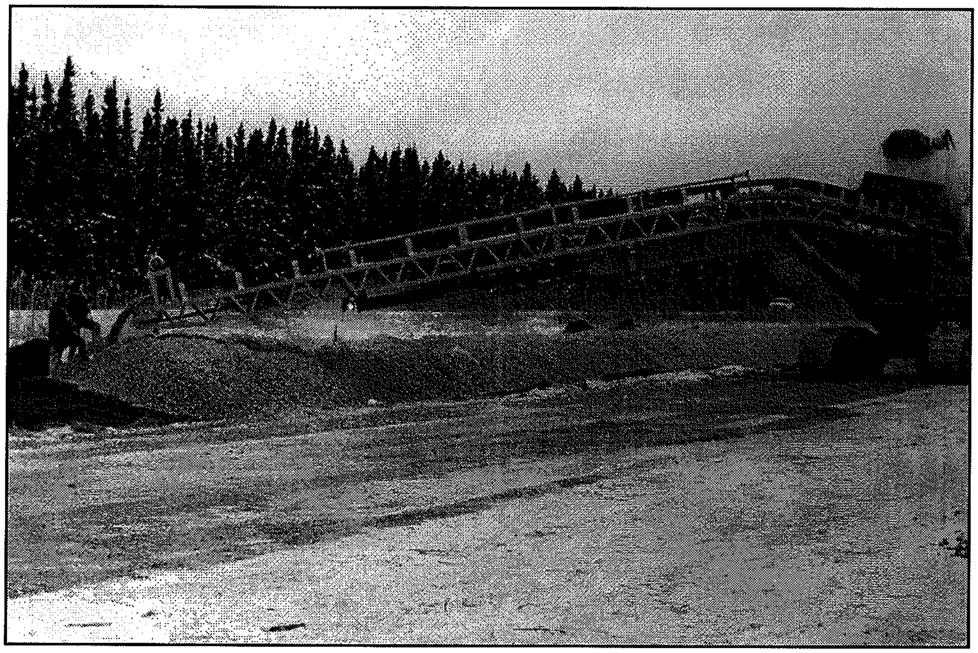


Figure 4-20. Placing granular ballast in the Boggy River Stretch (Mile 85.0 to Mile 88.5). Construction was completed during the winter season due to difficult site access.

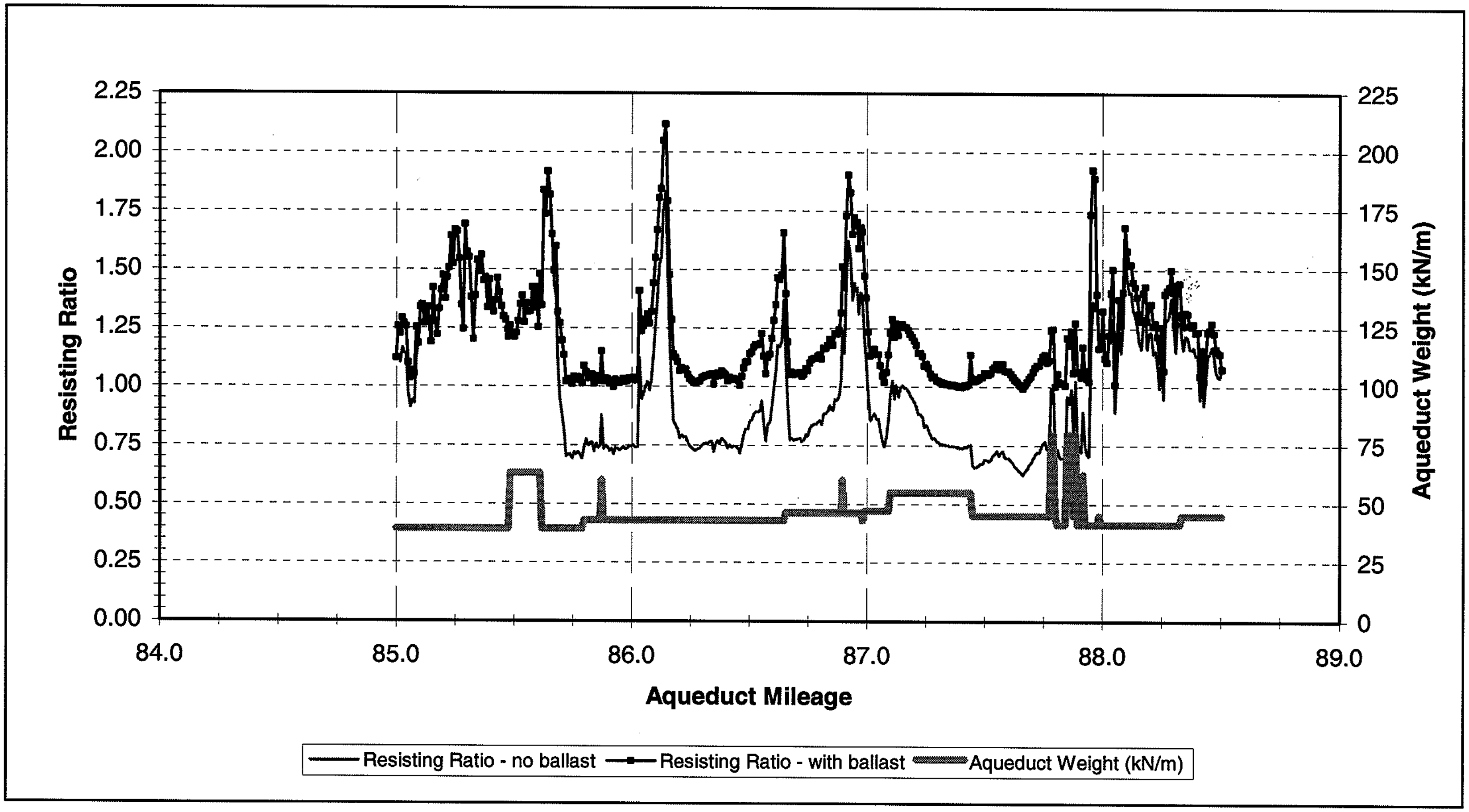


Figure 5-1. Resisting ratio profiles with and without granular ballast determined using the partial safety factors from the OHBDC (1991).

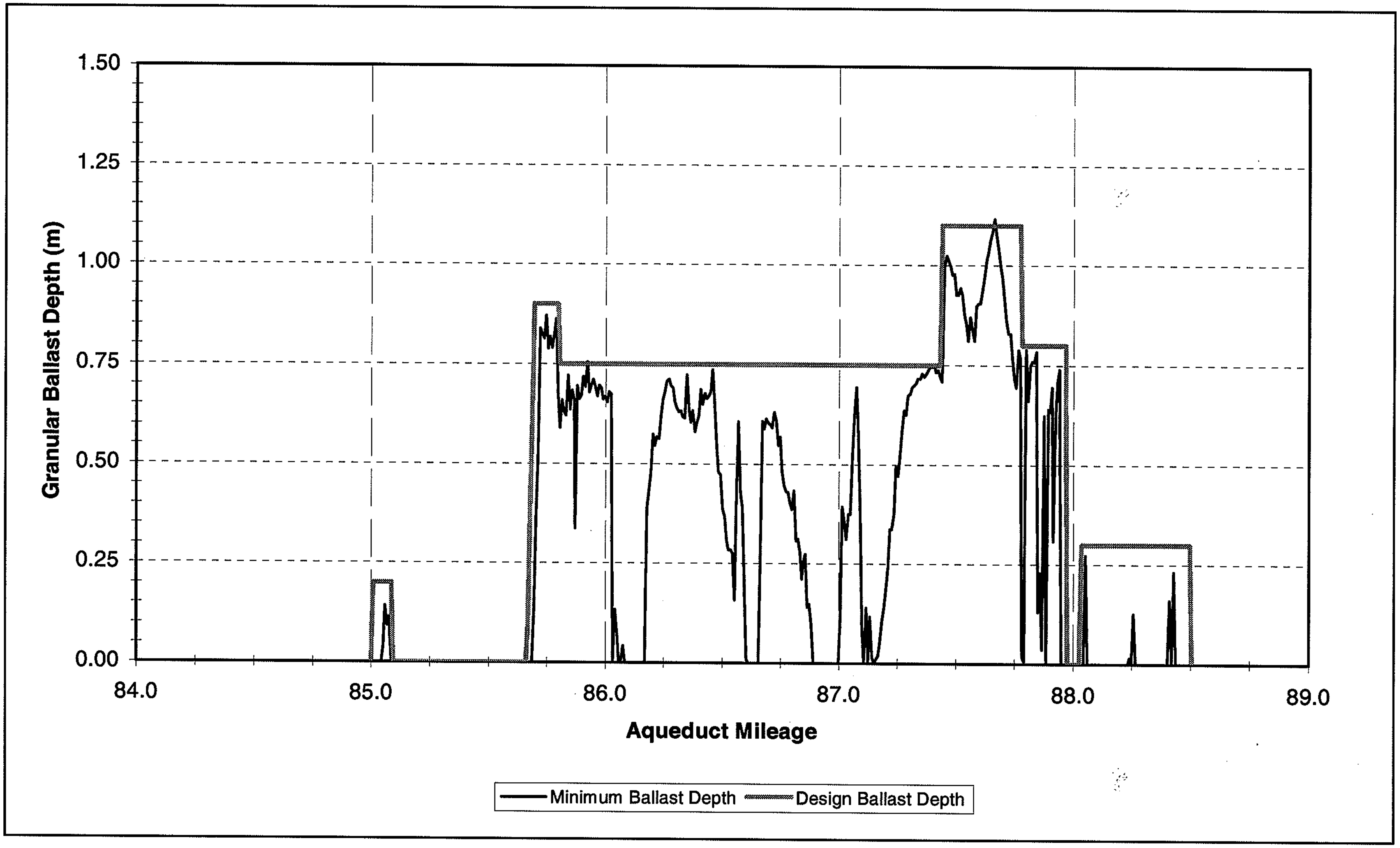


Figure 5-2. Profiles of the depths of granular ballast (minimum and design) determined using the partial safety factors from the OHBDC (1991).

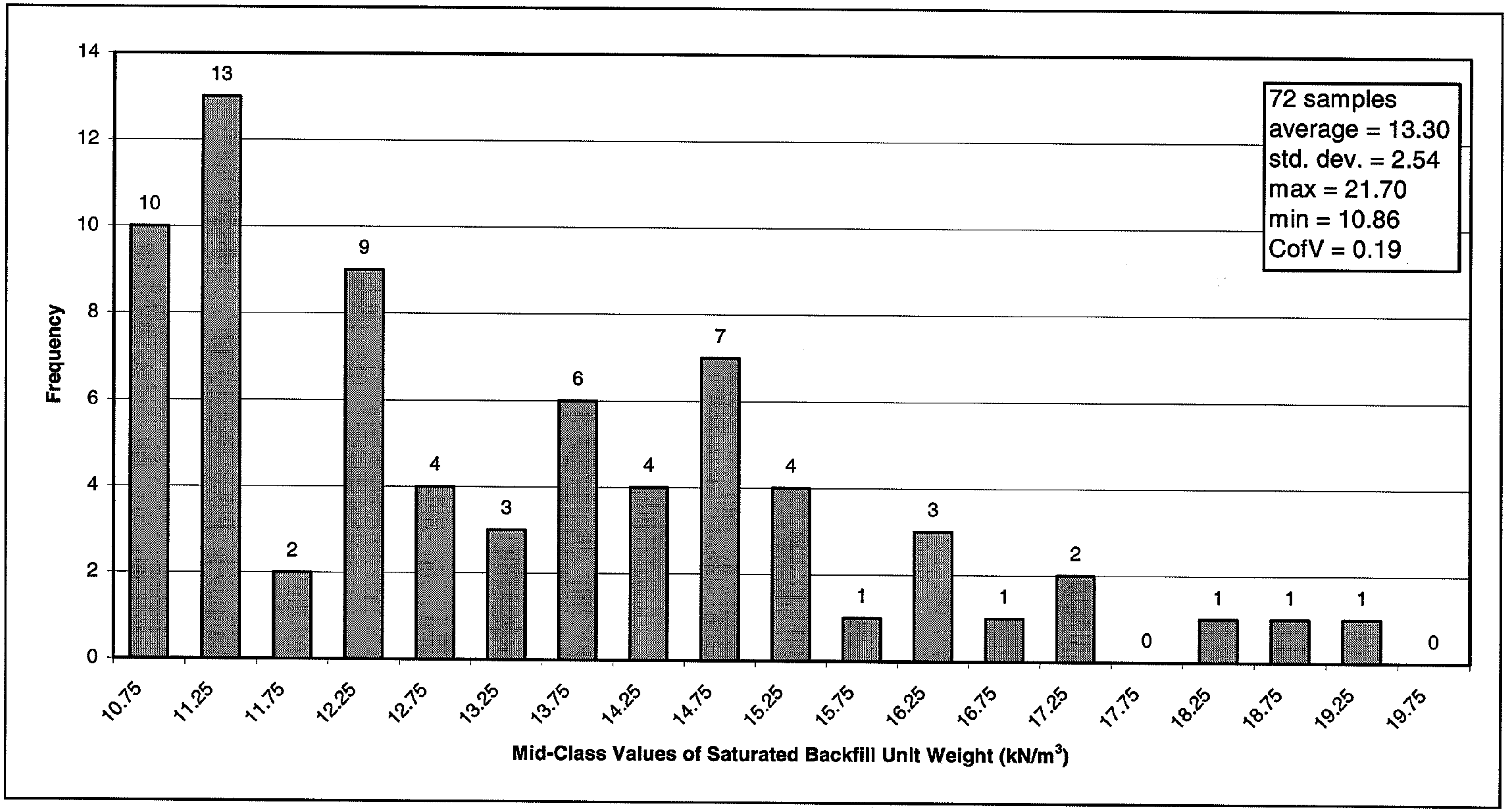


Figure 5-3. Histogram of measured and predicted saturated backfill unit weights used in calculating resistance factors for the unit weight of backfill.

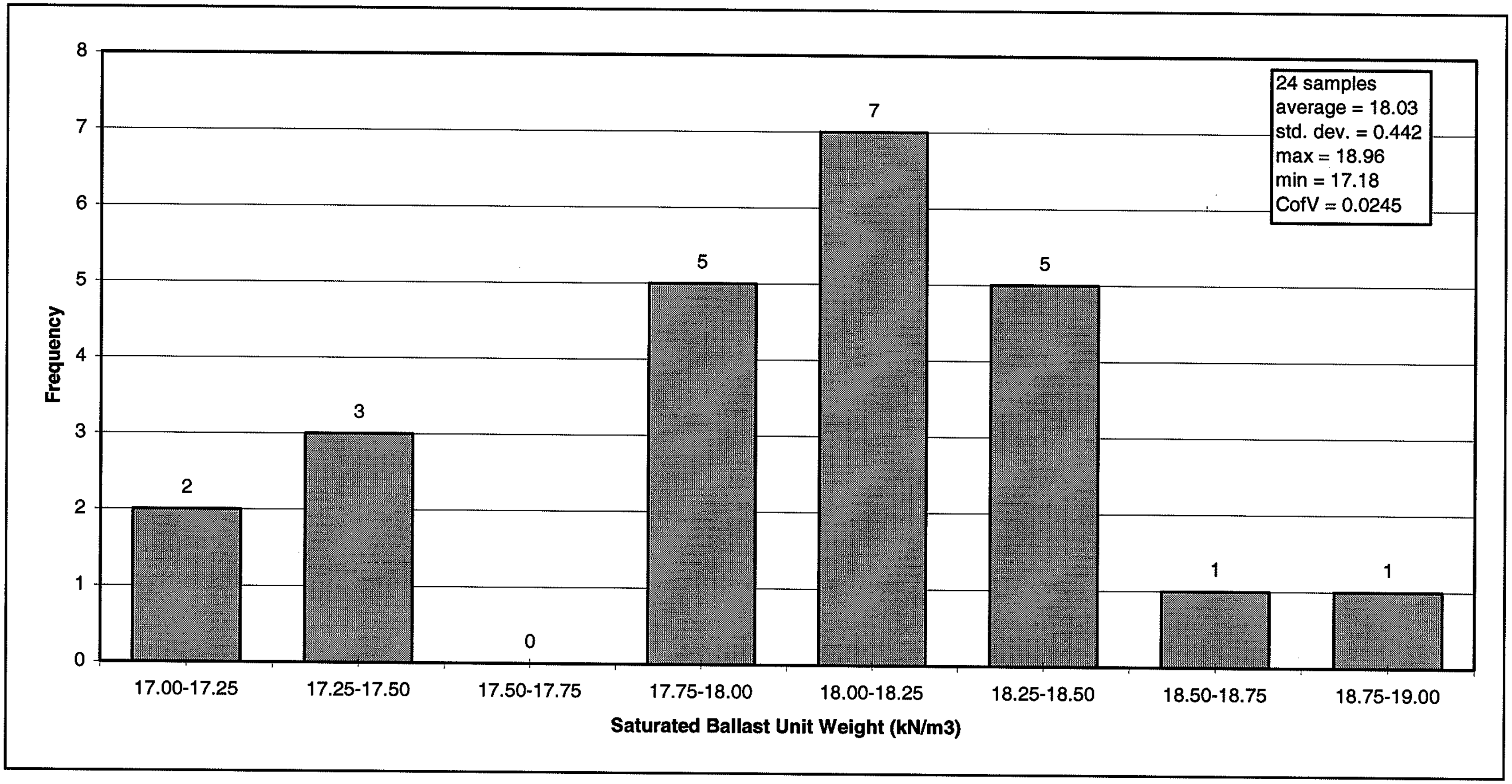


Figure 5-4. Histogram of measured unit weights of saturated granular ballast used in calculating resistance factors for the unit weight of ballast.

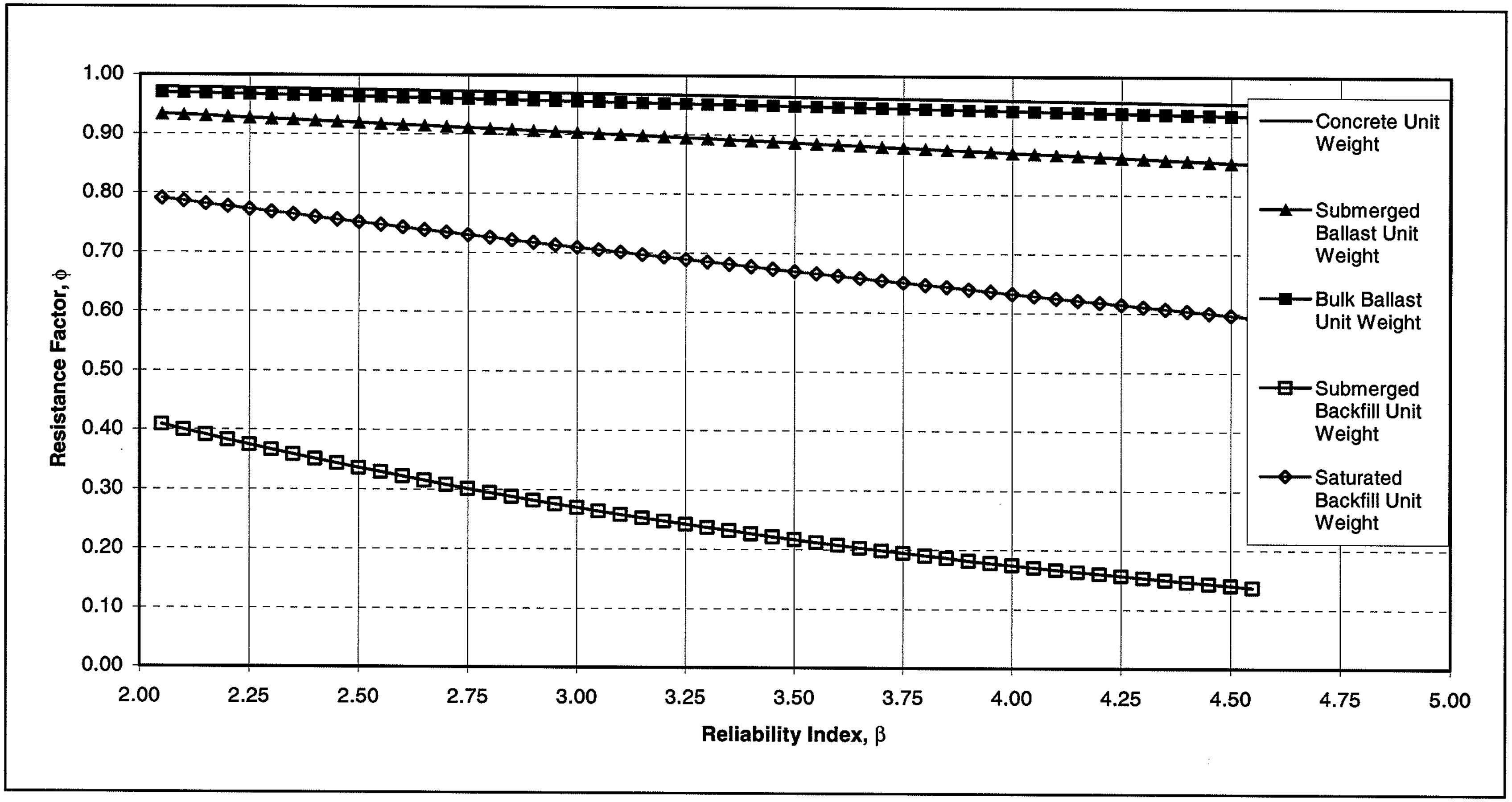


Figure 5-5. Relationship between the reliability index, β and the calculated resistance factor, ϕ for each of the variables included in the buoyancy analysis.

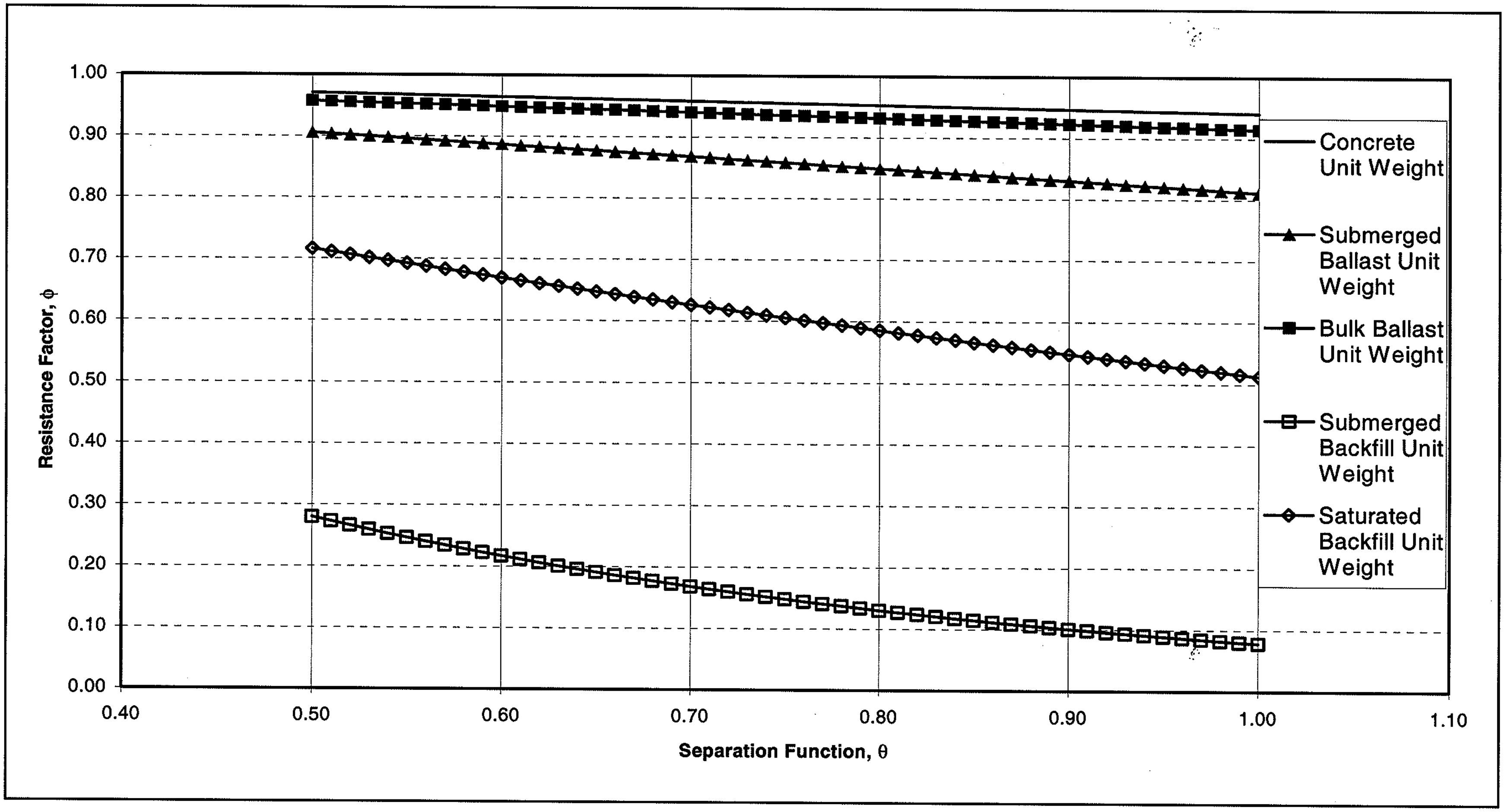


Figure 5-6. Relationship between the separation function, θ and the calculated resistance factor, φ for the each of the variables included in the buoyancy analysis.

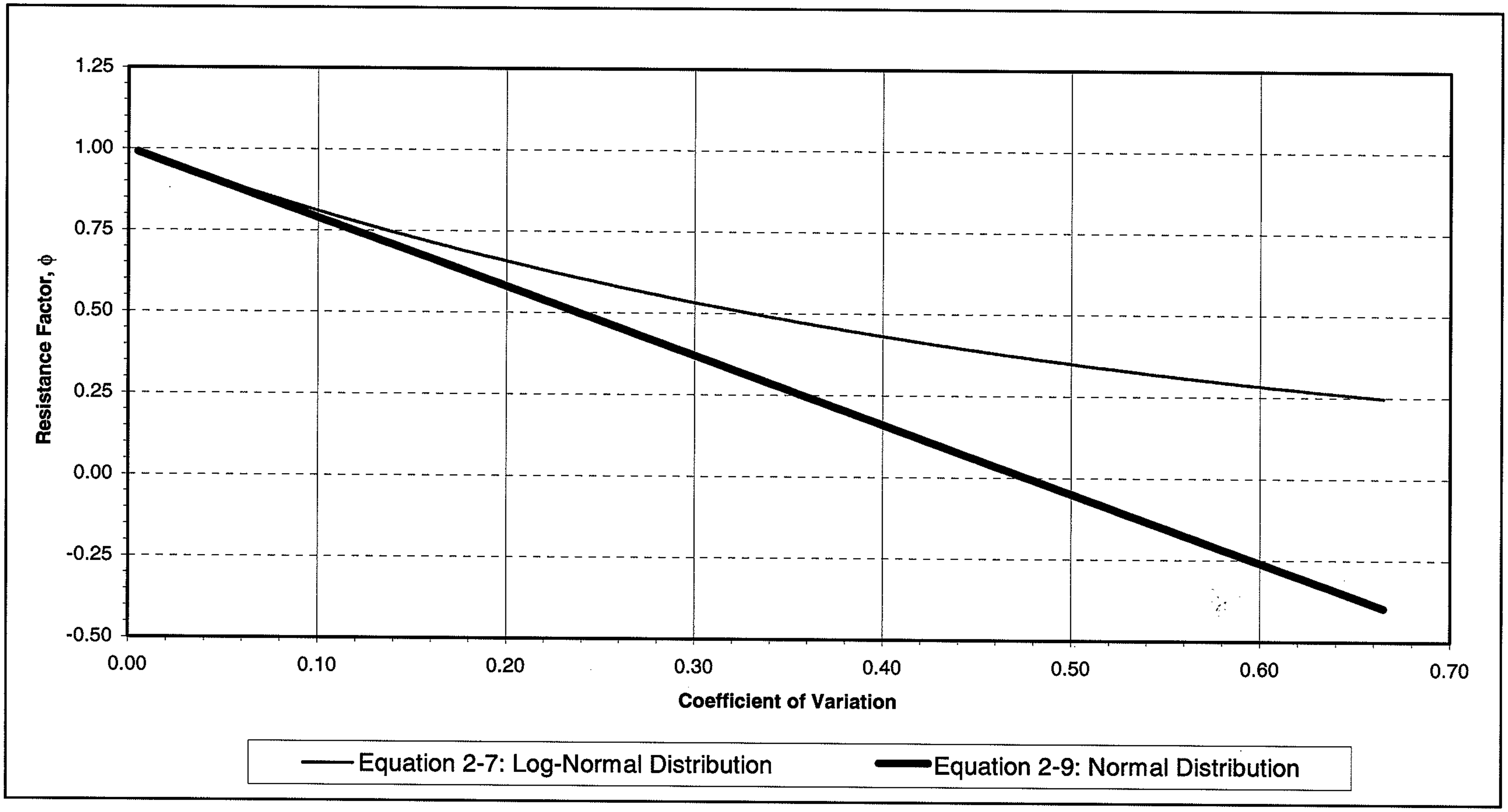


Figure 5-7. Relationship between the coefficient of variation and the resistance factor, ϕ calculated using Equations 2-7 and 2-9.

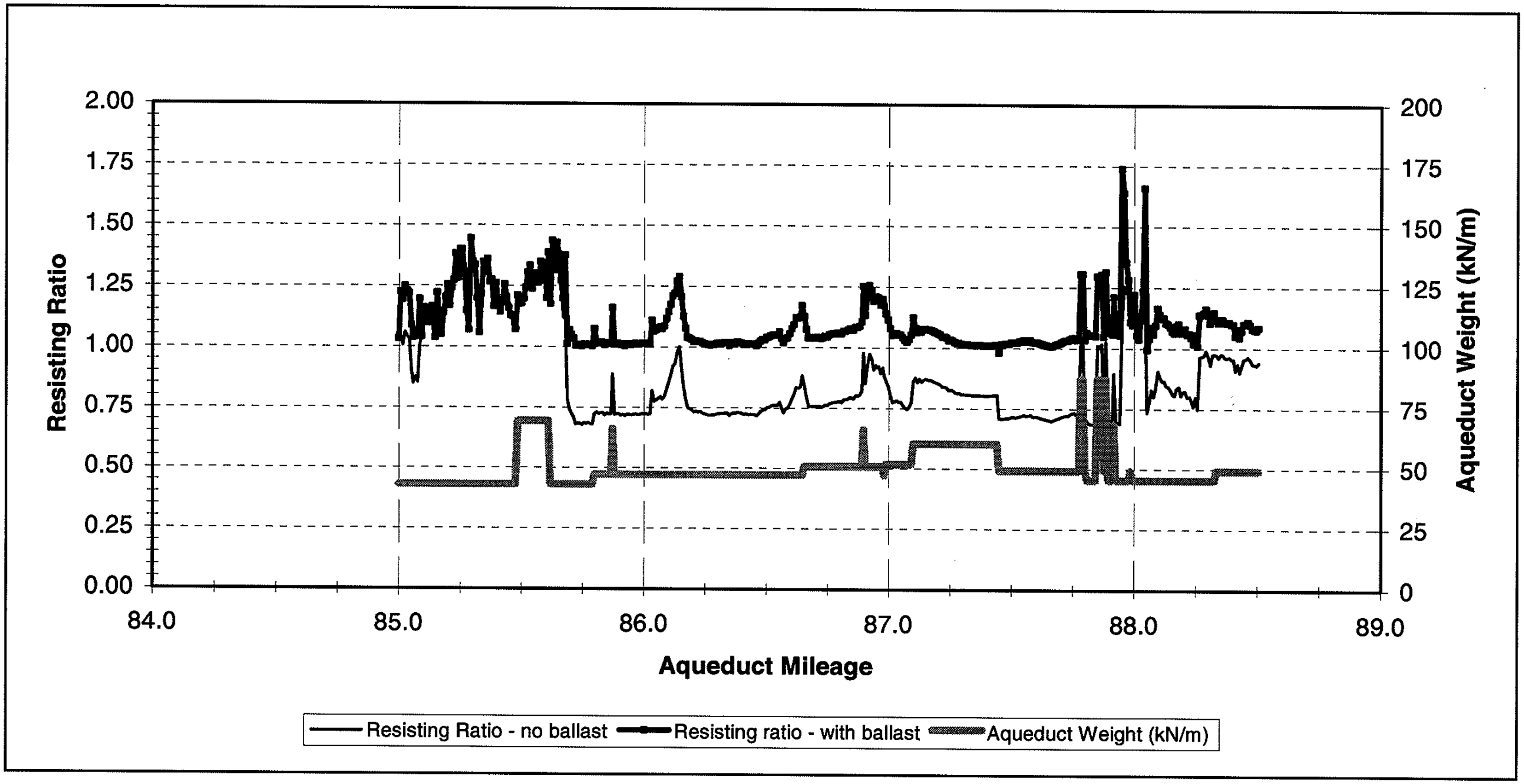


Figure 5-8. Resisting ratio profiles with and without granular ballast determined using project-specific (P-S) partial safety factors.

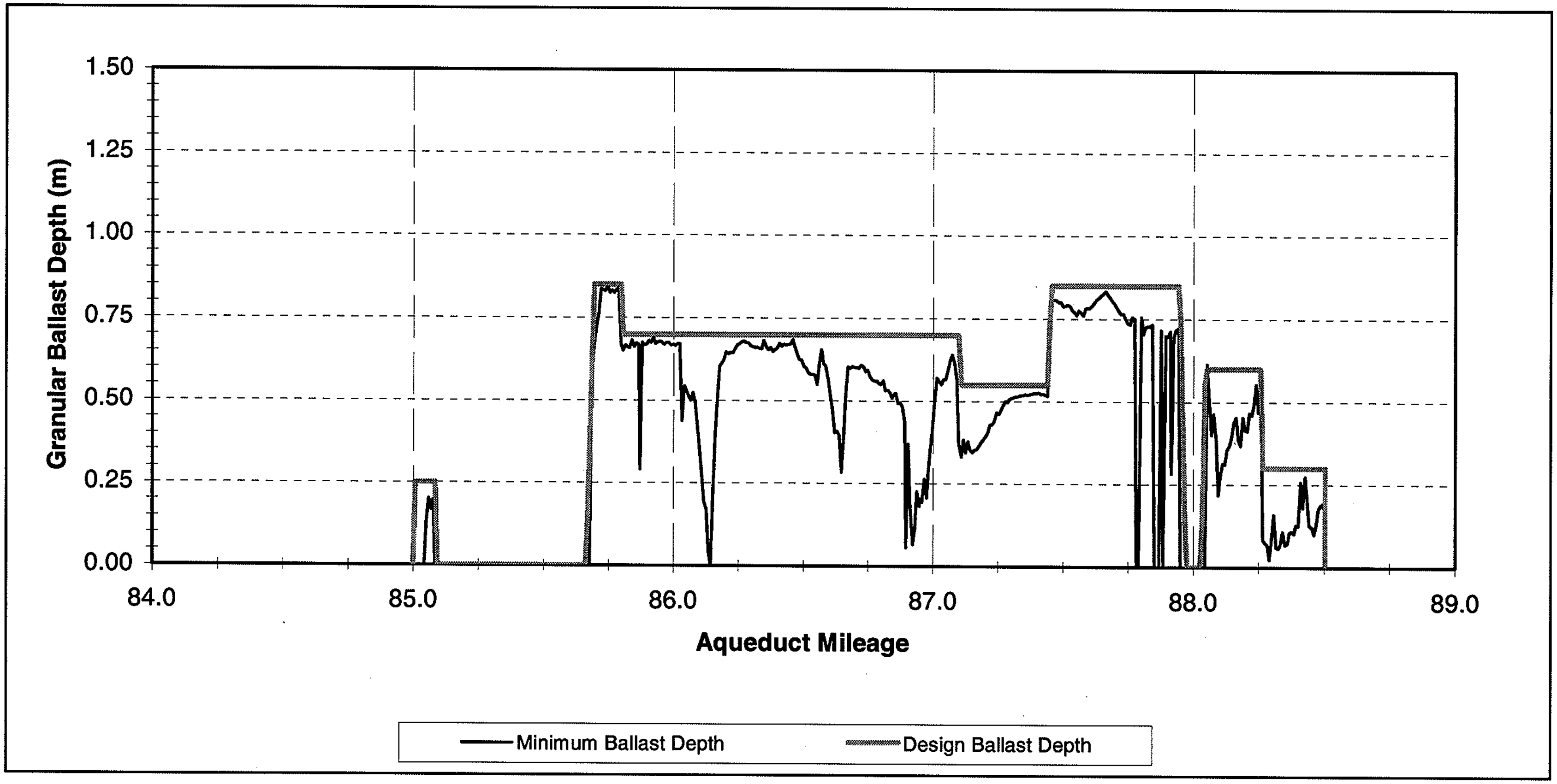


Figure 5-9. Profiles of the depths of granular ballast (minimum and design) determined using project-specific (P-S) partial safety factors.

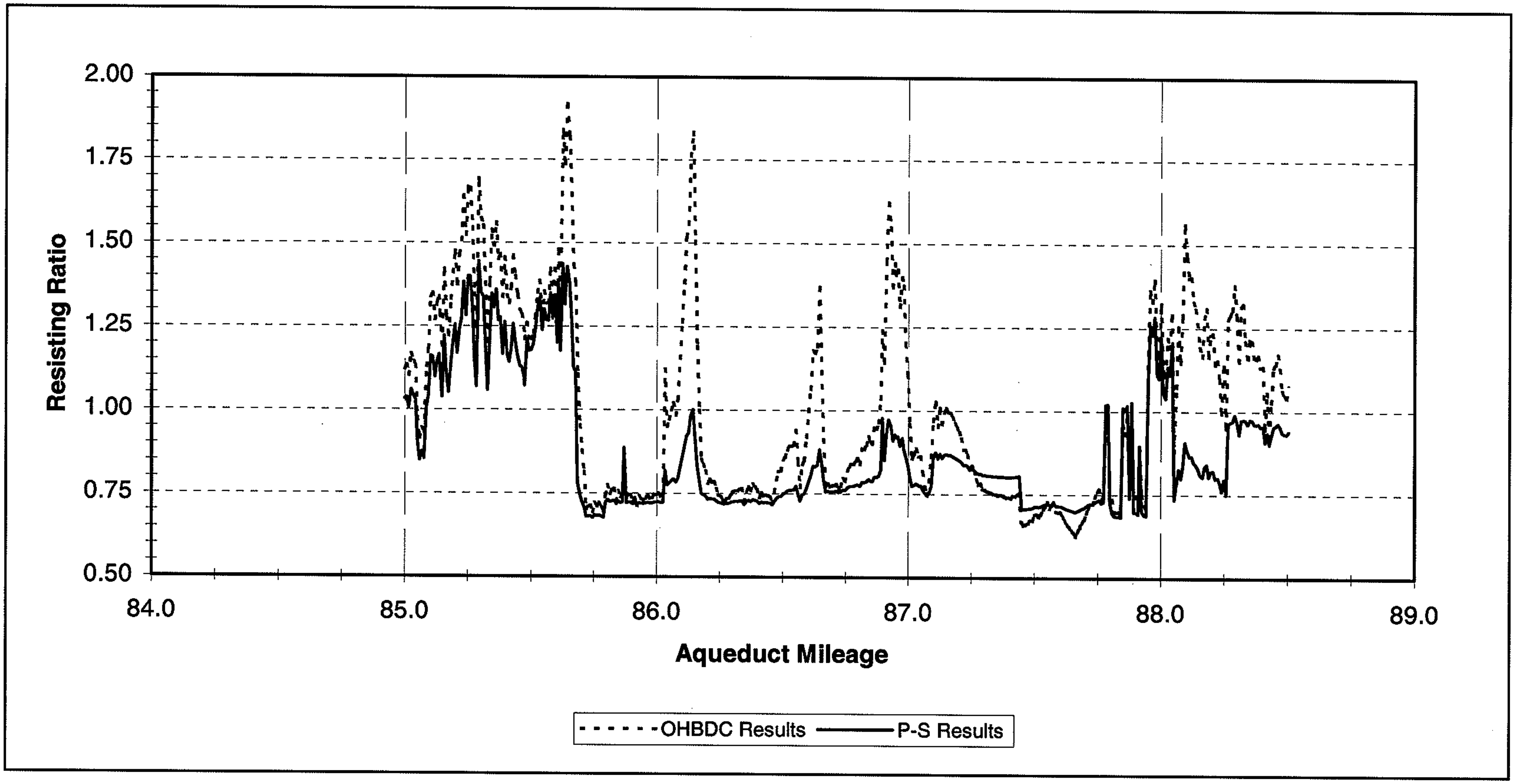


Figure 5-10. Comparison of the resisting ratio profiles from the two buoyancy analyses completed using partial safety factors.

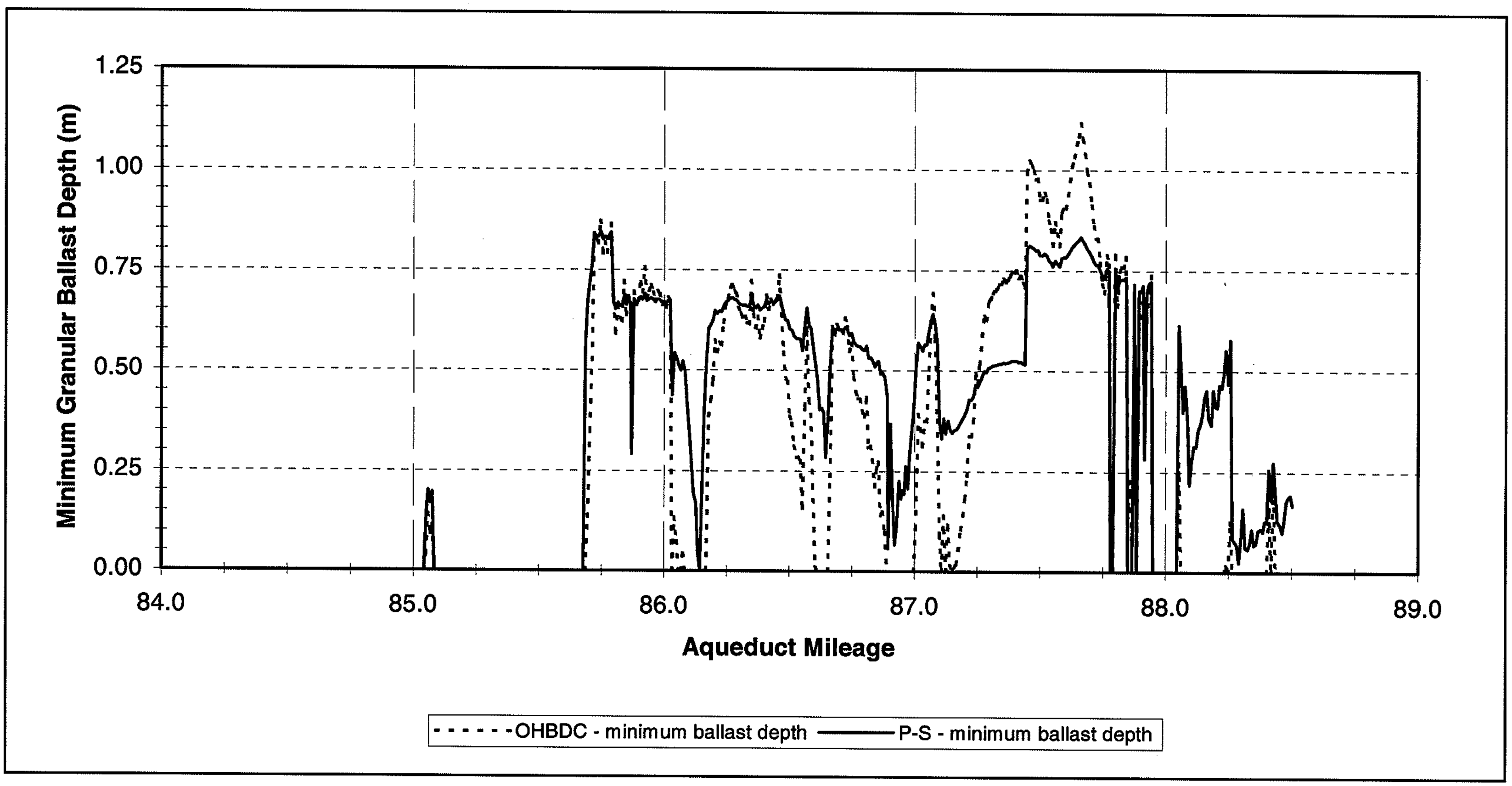


Figure 5-11. Comparison of the profiles of the minimum depths of granular ballast determined from the two buoyancy analyses completed using partial safety factors.

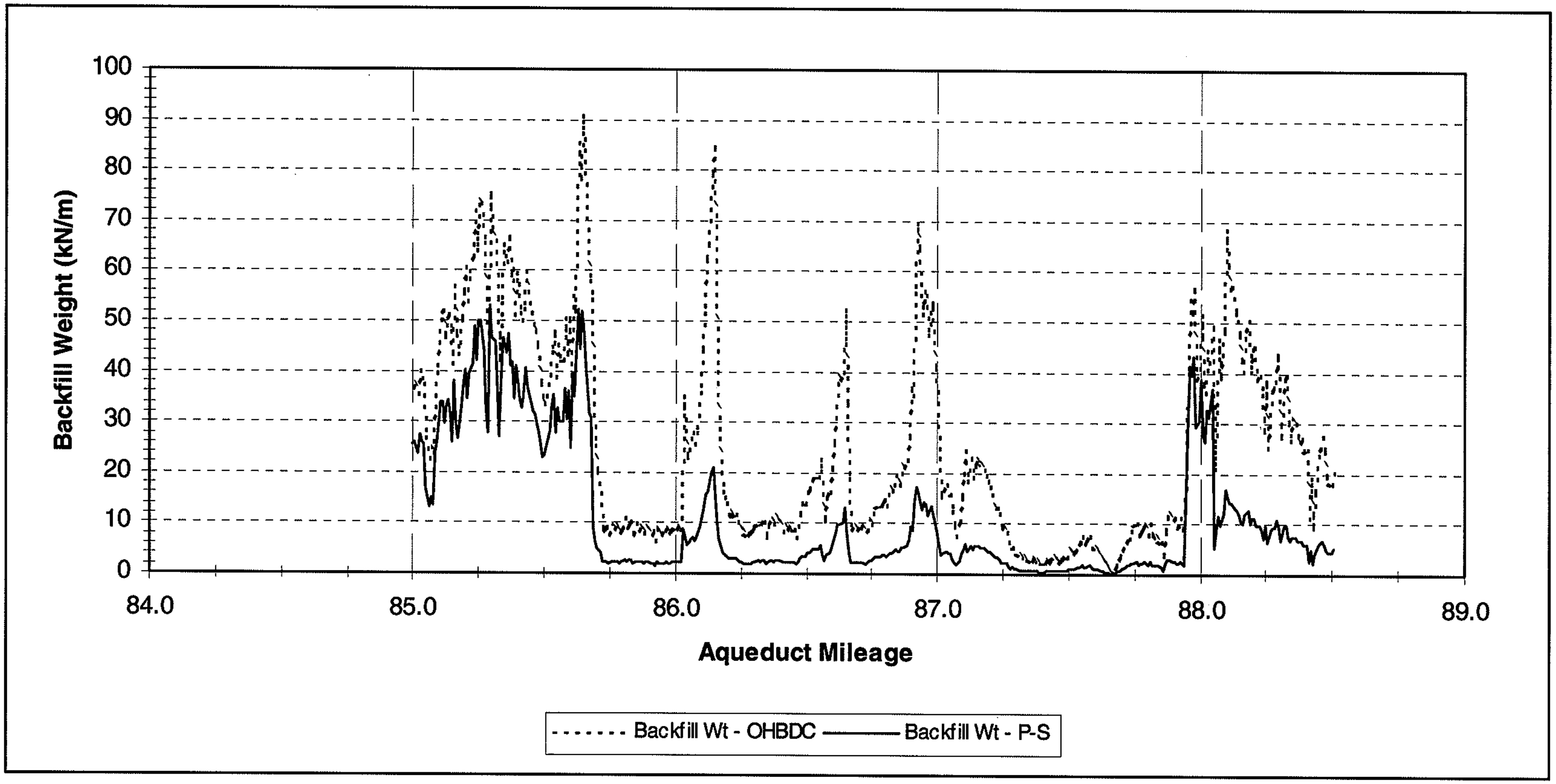


Figure 5-12. Comparison of the profiles for the backfill weights calculated using the project-specific (P-S) partial safety factors and those from the OHBDC (1991).

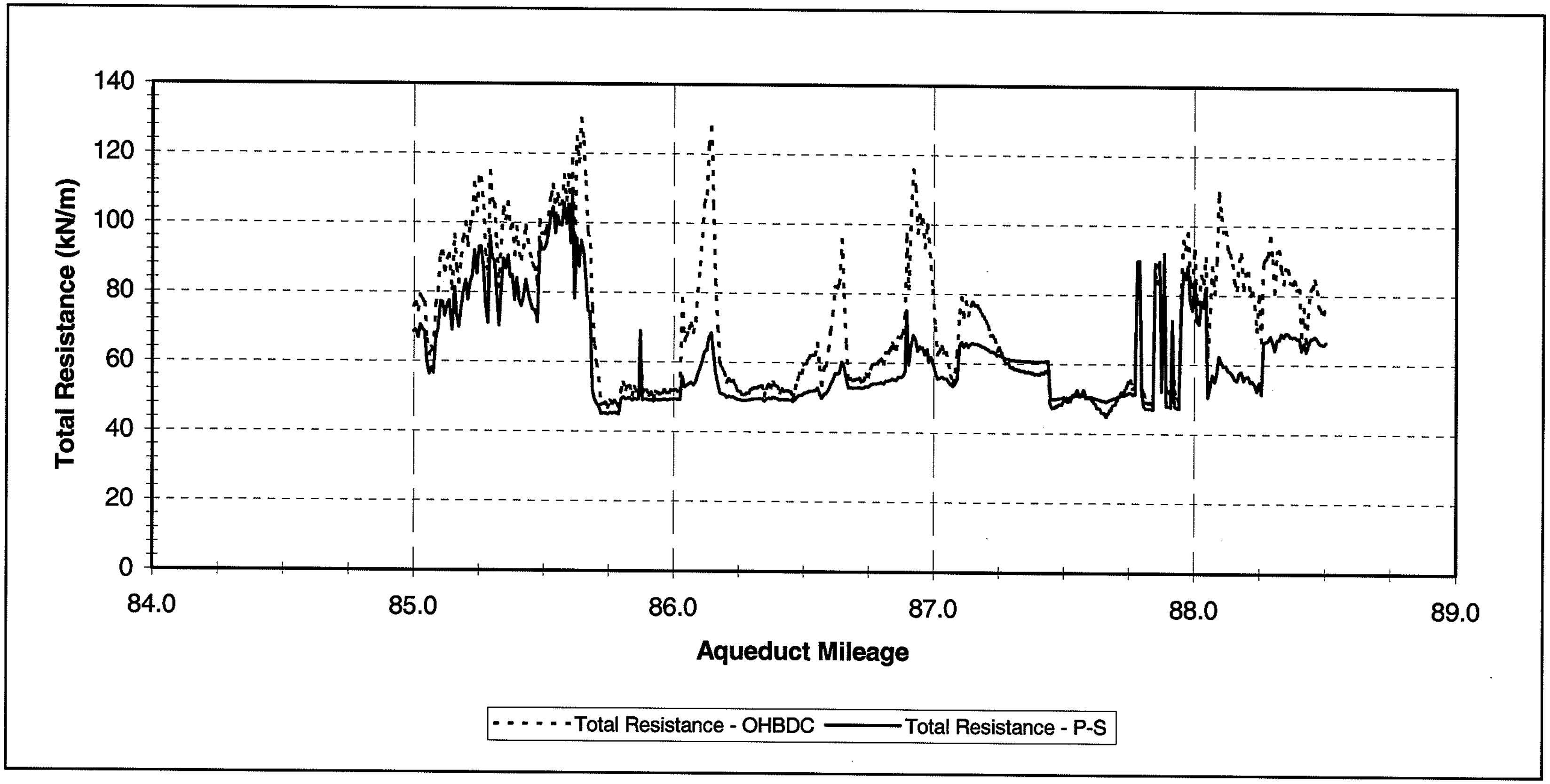


Figure 5-13. Comparison of the profiles for the total resistance calculated using the project-specific (P-S) partial safety factors and those from the OHBDC (1991).

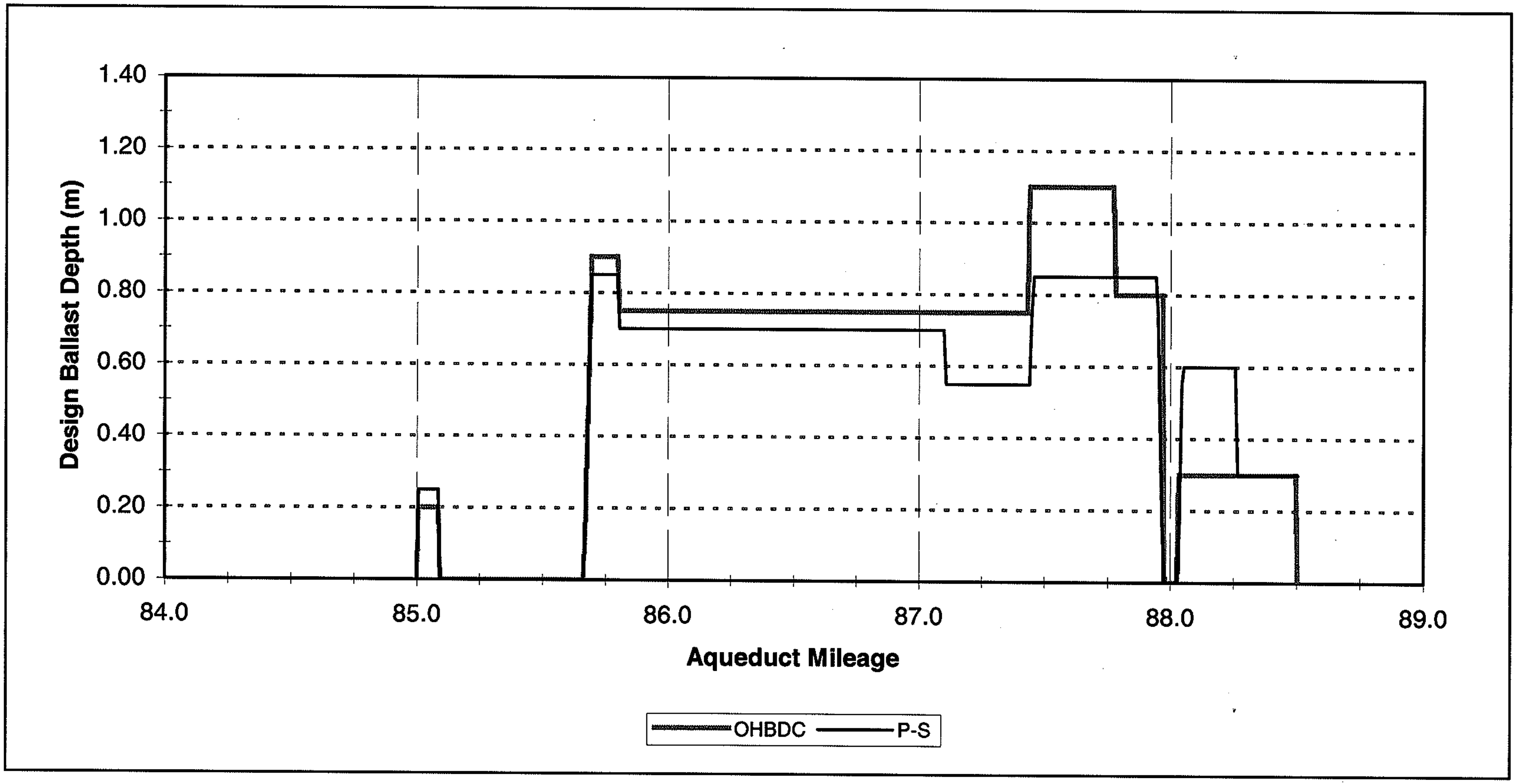


Figure 5-14. Comparison of the profiles for the depths of granular ballast selected for final design. The design depths were based on the minimum depths of granular ballast that were calculated using the project-specific (P-S) partial safety factors and those from the OHBDC (1991).

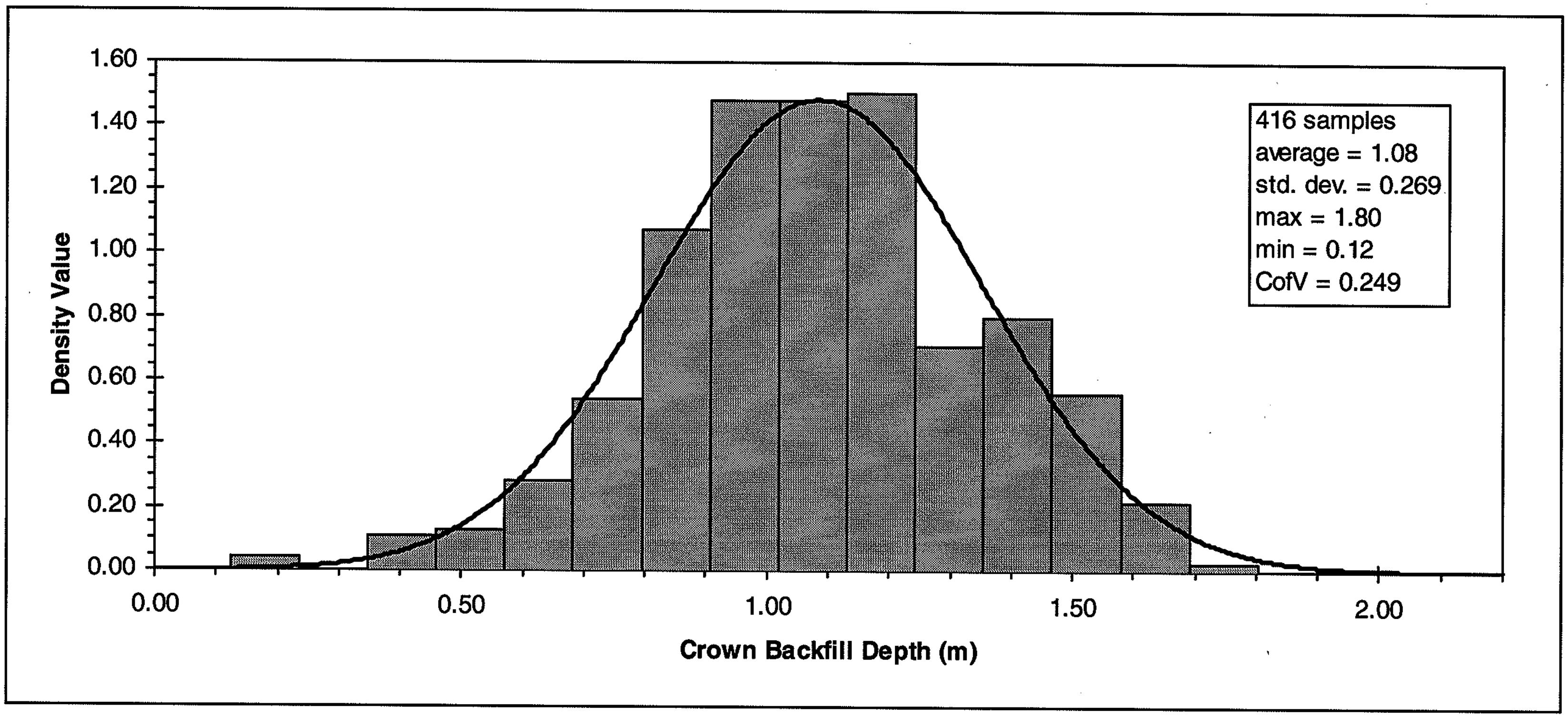


Figure 5-15. Normal probability distribution function fitted to data for the depths of crown backfill.

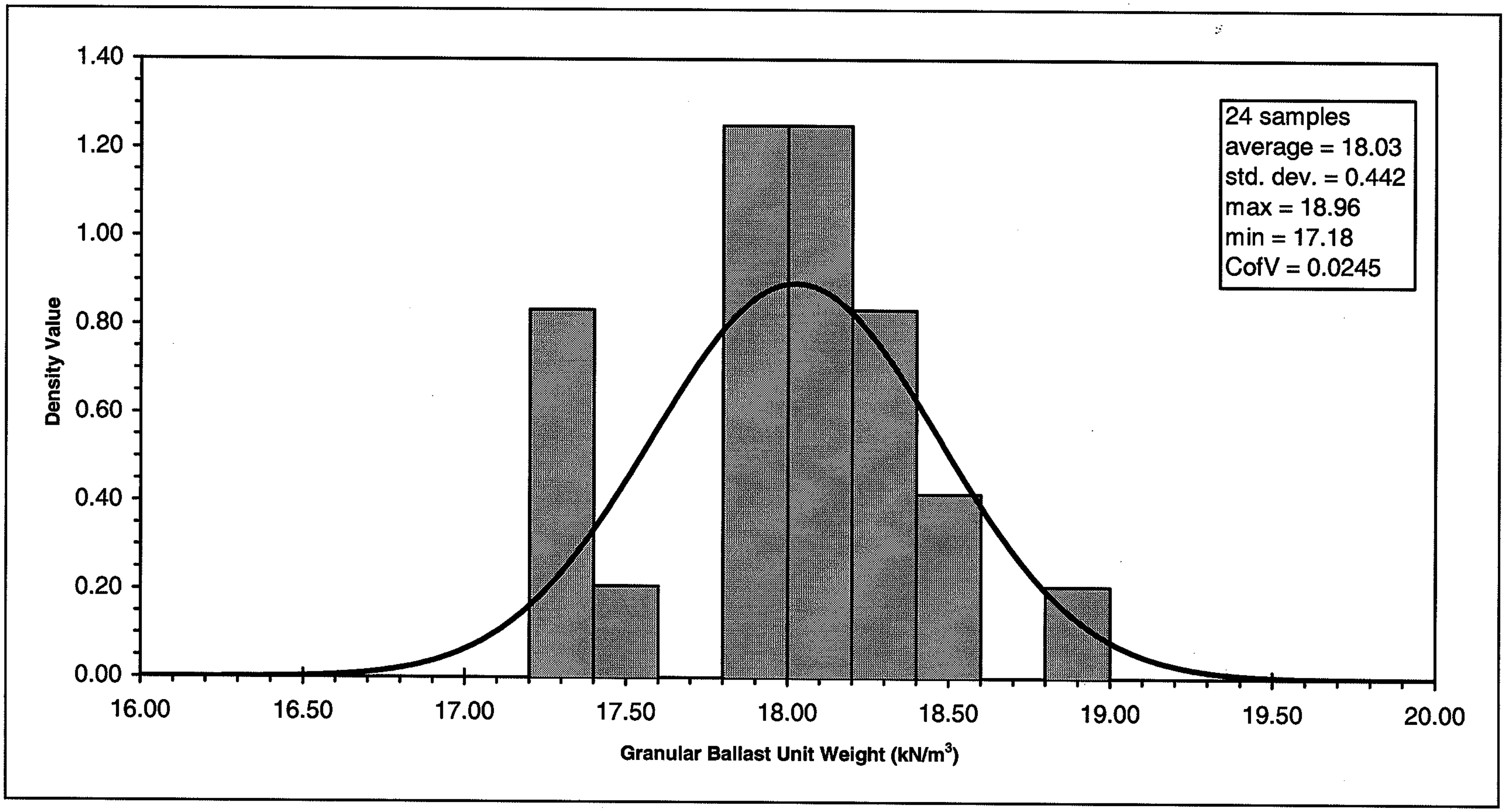


Figure 5-16. Normal probability distribution function fitted to data for the unit weight of granular ballast.

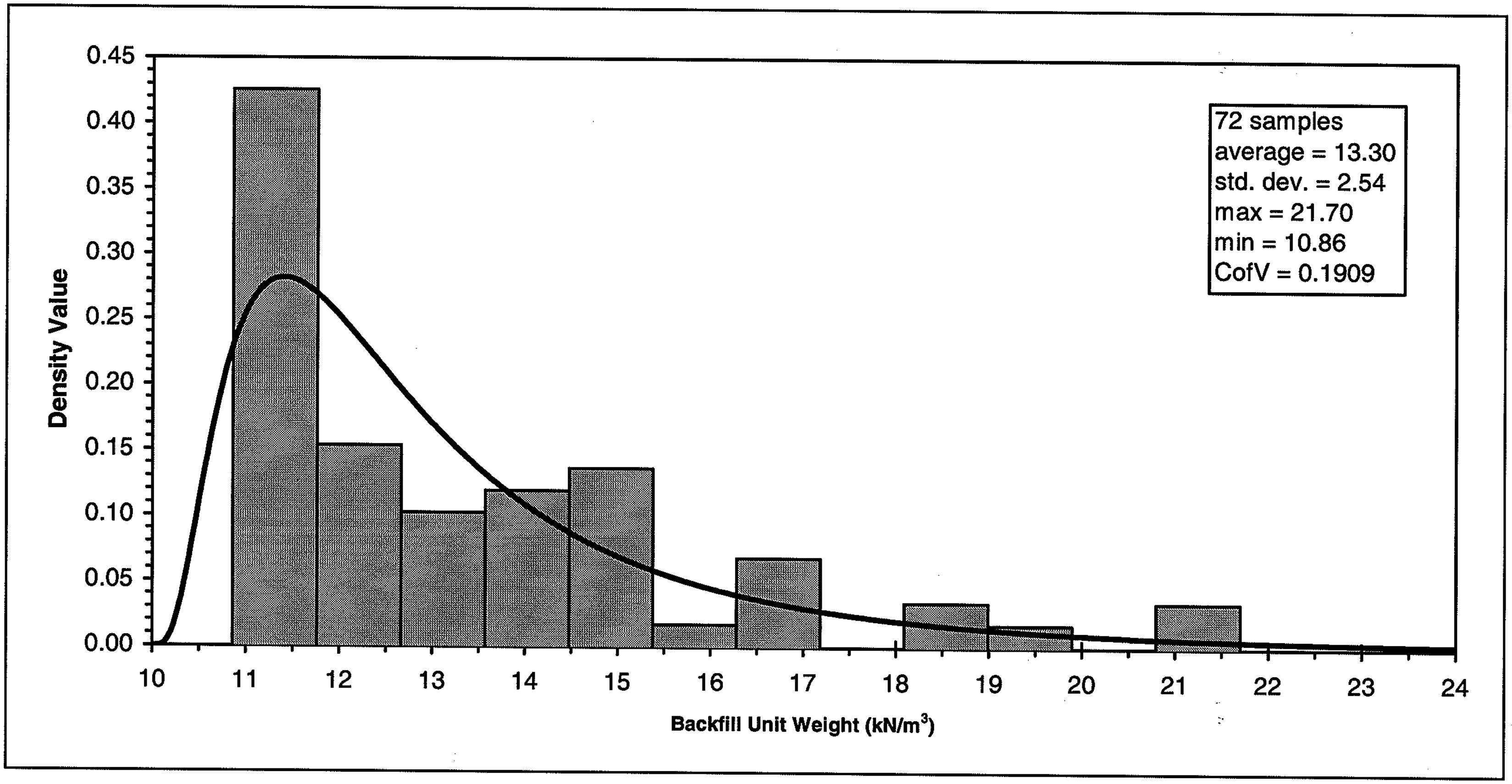


Figure 5-17. Log-normal probability distribution function fitted to data for the unit weight of backfill.

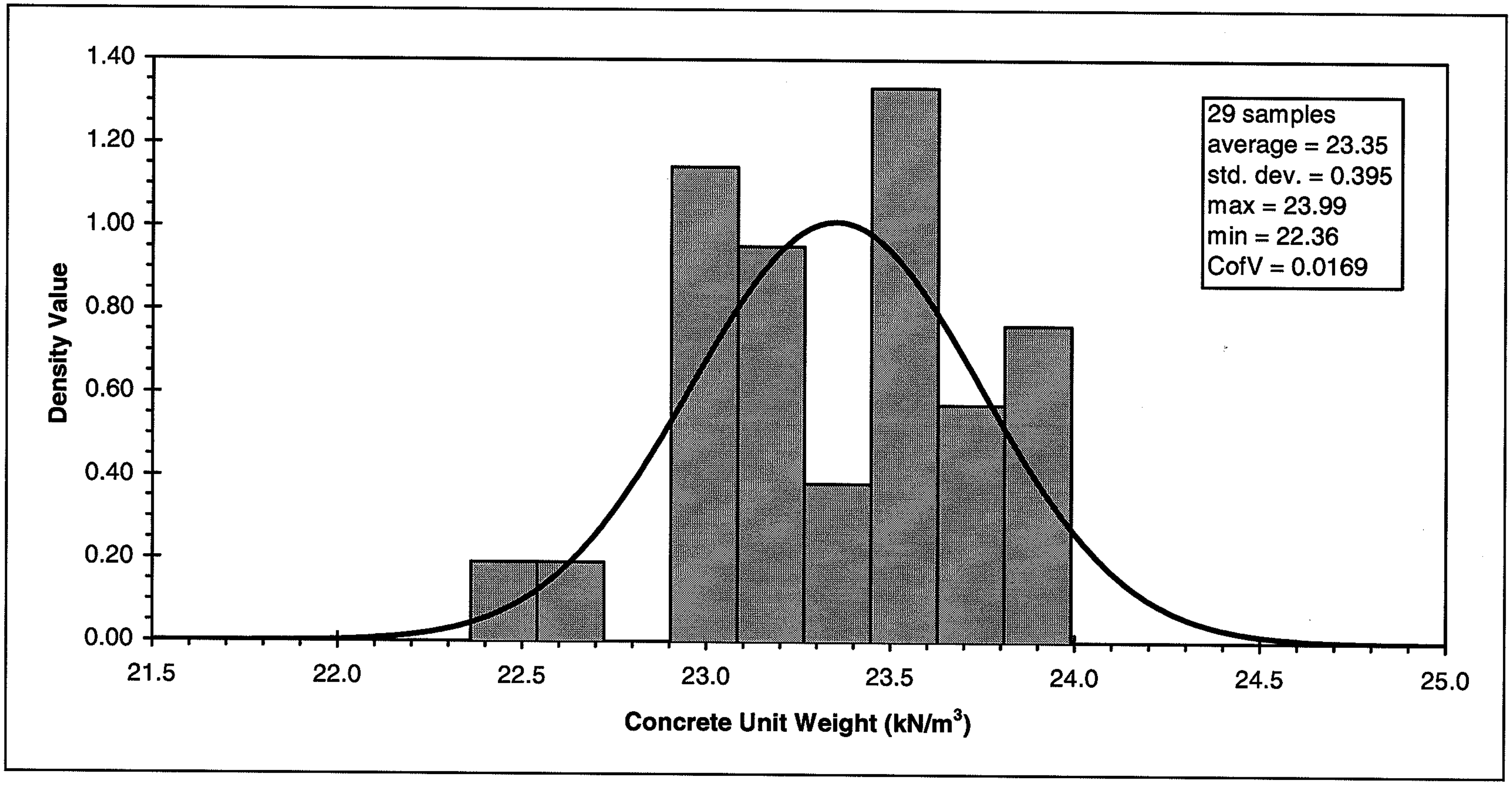


Figure 5-18. Normal probability distribution function fitted to data for the unit weight of concrete.

SHOAL LAKE AQUEDUCT BUOYANCY ASSESSMENT
Monte Carlo Simulation Model
BP Aqueduct Section (no shear resistance)

Target Reliability = 1.1×10^{-4} (0.011%)

Input Variables:				**Distribution Type**
Saturated backfill unit wt. =	12.95	kN/m^3	*input*	Log-Normal Dist
Backfill depth =	1.08	m	*input*	Normal distribution
Shear strength =	0.00	kPa	*input*	n/a
Compacted depth =	0.00	m	*input*	n/a
Concrete unit weight =	23.35	kN/m^3	*input*	Normal distribution
Concrete area =	2.13	m^2	*input*	Triangular distribution
Total section area =	6.76	m^2	*input*	Triangular distribution
Saturated ballast unit weight =	18.02	kN/m^3	*input*	Normal distribution
Fixed Values:				
Maximum aqueduct width =	4.06	m		
Archleg backfill volume =	4.11	m^3/m		
Unit weight of water =	9.81	kN/m^3		
Disturbing Force:				
Buoyant Force =	**66.36**	kN/m		
Resisting Forces:				
Crown backfill wt =	13.83	kN/m	*output*	
Arch leg backfill wt =	12.89	kN/m	*output*	
Shear resistance =	0.00	kN/m	*output*	
Aqueduct self wt =	49.73	kN/m	*output*	
Total Resisting Force =	**76.45**	kN/m	*output*	
Resisting Ratio =	**1.15**	Total Resist. Force / Buoyant Force	*output*	
Additional Resisting Force (granular ballast):				
Minimum depth of ballast required =	0.00	m	*output*	
New Total Resisting Force =	**76.45**	kN/m	*output*	
New Resisting Ratio =	**1.15**	Total Resist. Force / Buoyant Force	*output*	

Figure 5-19. Typical set up of a monte carlo simulation model used for the buoyancy analysis.

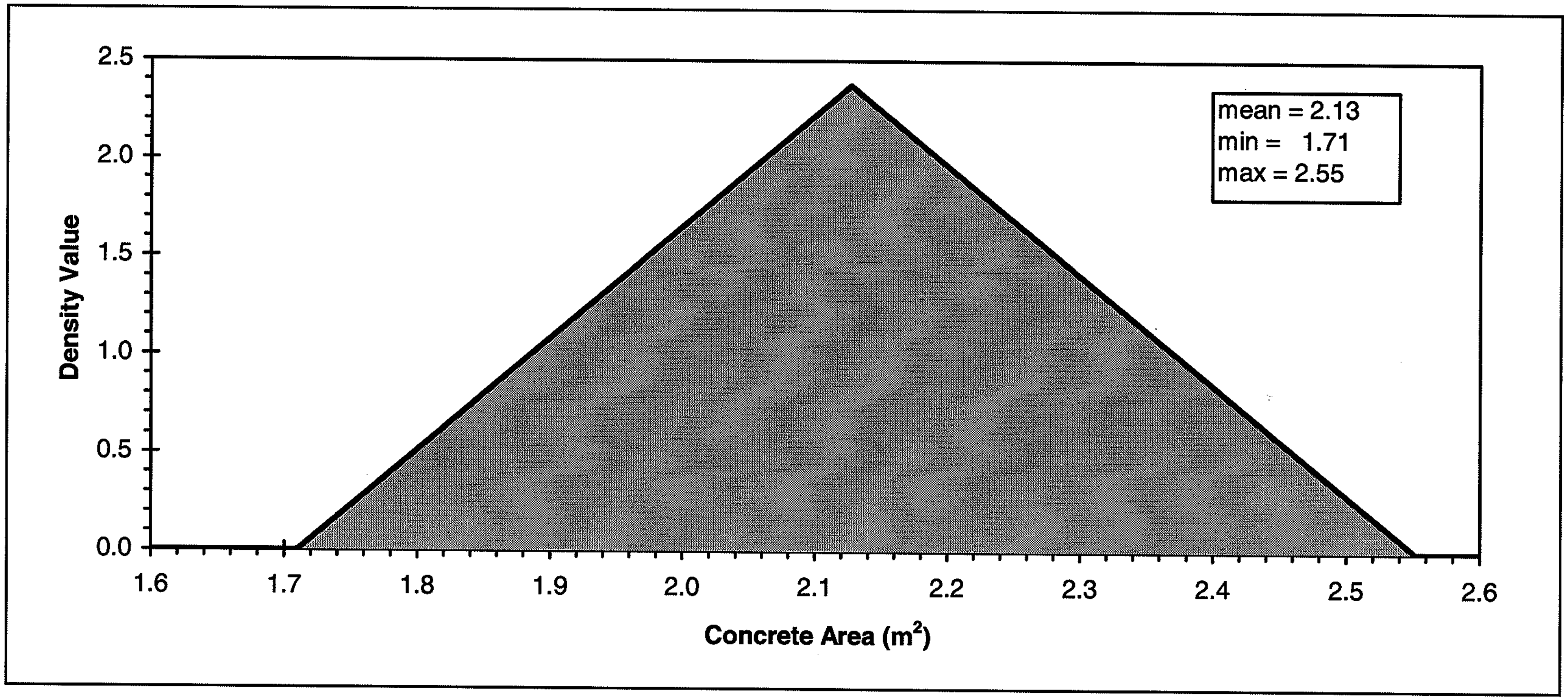


Figure 5-20. Typical triangular probability distribution function used to estimate the variability of concrete area for each aqueduct section analyzed using monte carlo simulation. This distribution is for the 'BP' aqueduct section.

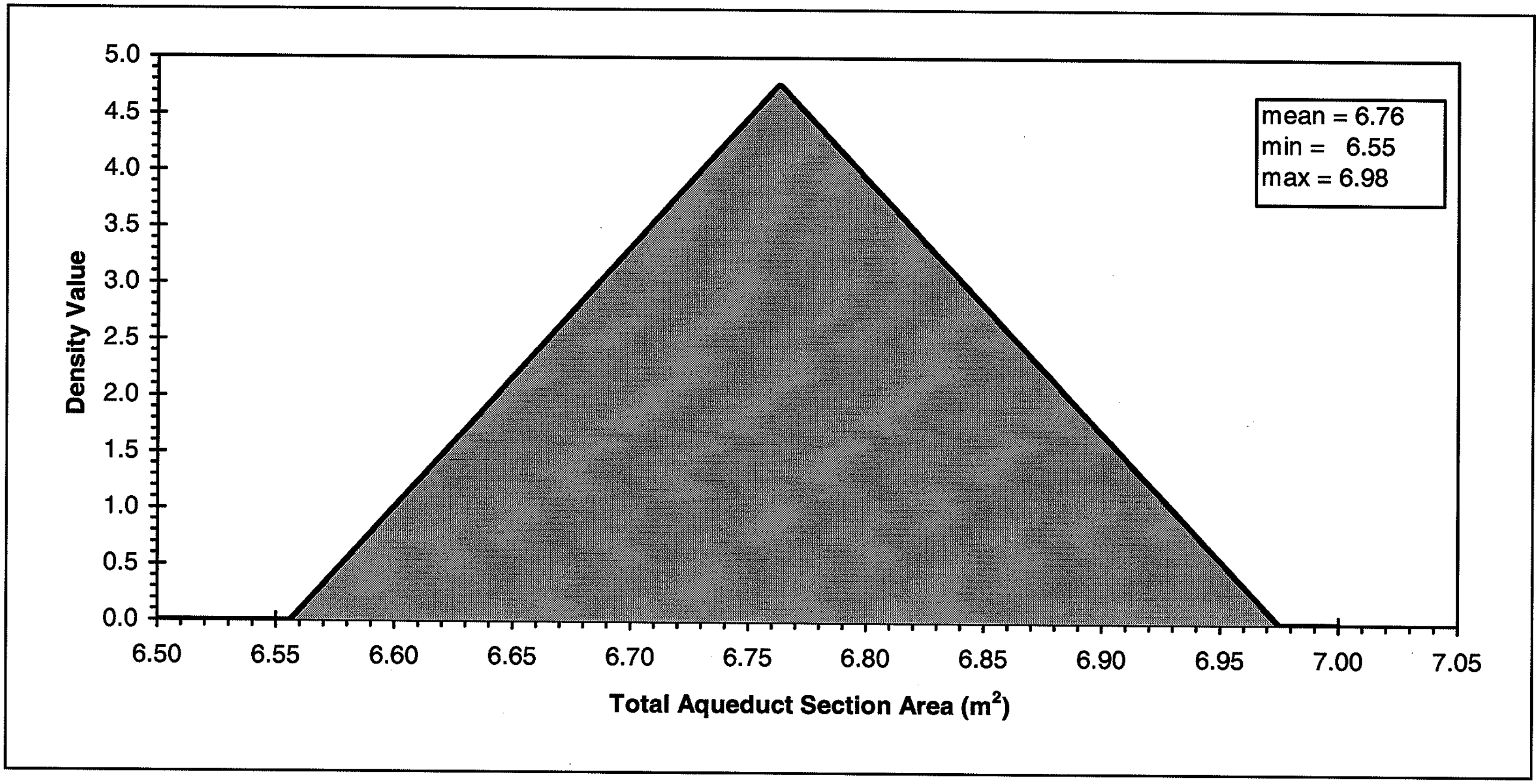


Figure 5-21. Typical triangular probability distribution function used to estimate the variability of total section area for each aqueduct section analyzed using monte carlo simulation. This distribution is for the 'BP' aqueduct section.

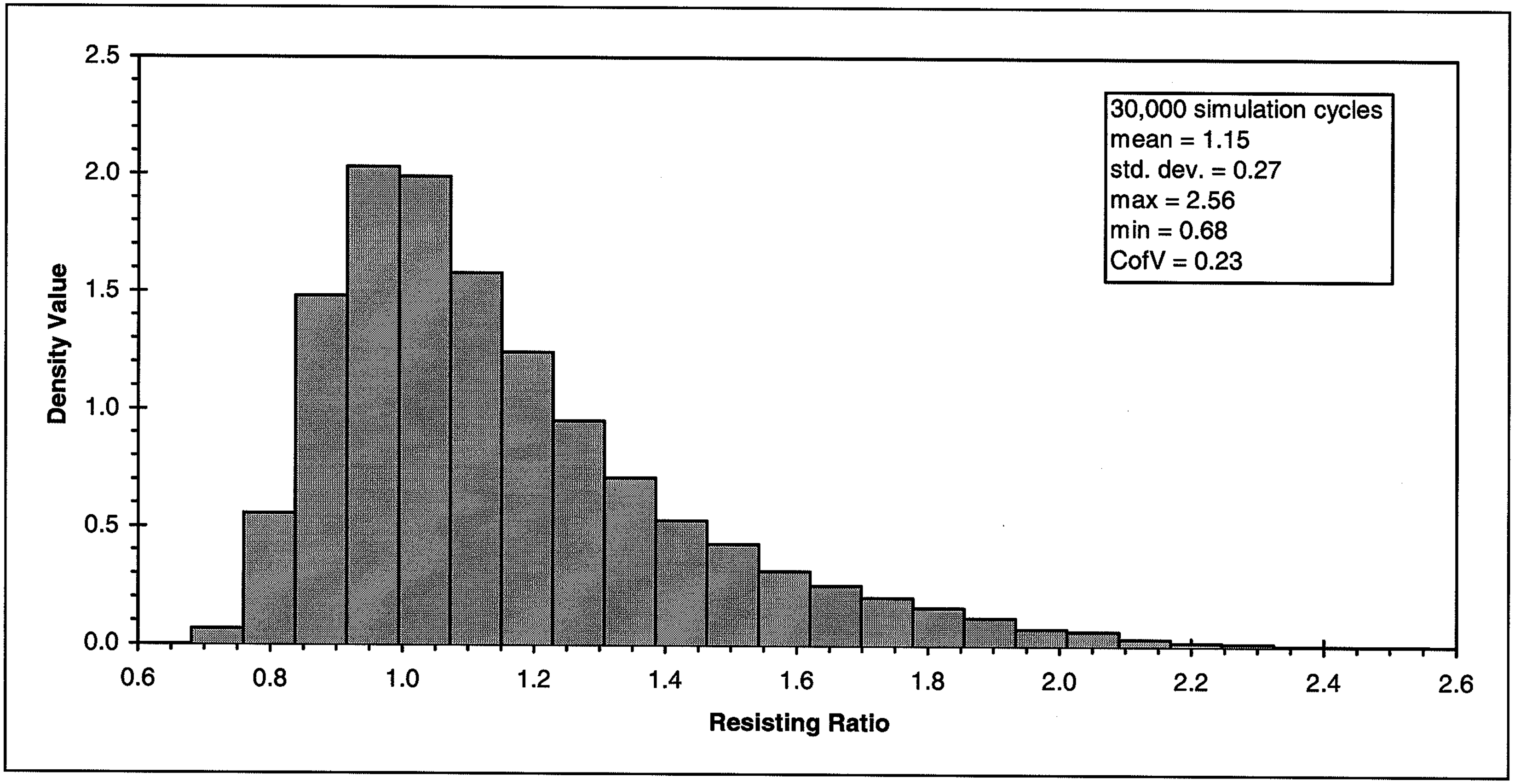


Figure 5-22. Distribution of resisting ratios calculated for the BP aqueduct section over 30,000 simulation cycles. The probability that the resisting ratio is less than 1.0 is 33.5 percent.

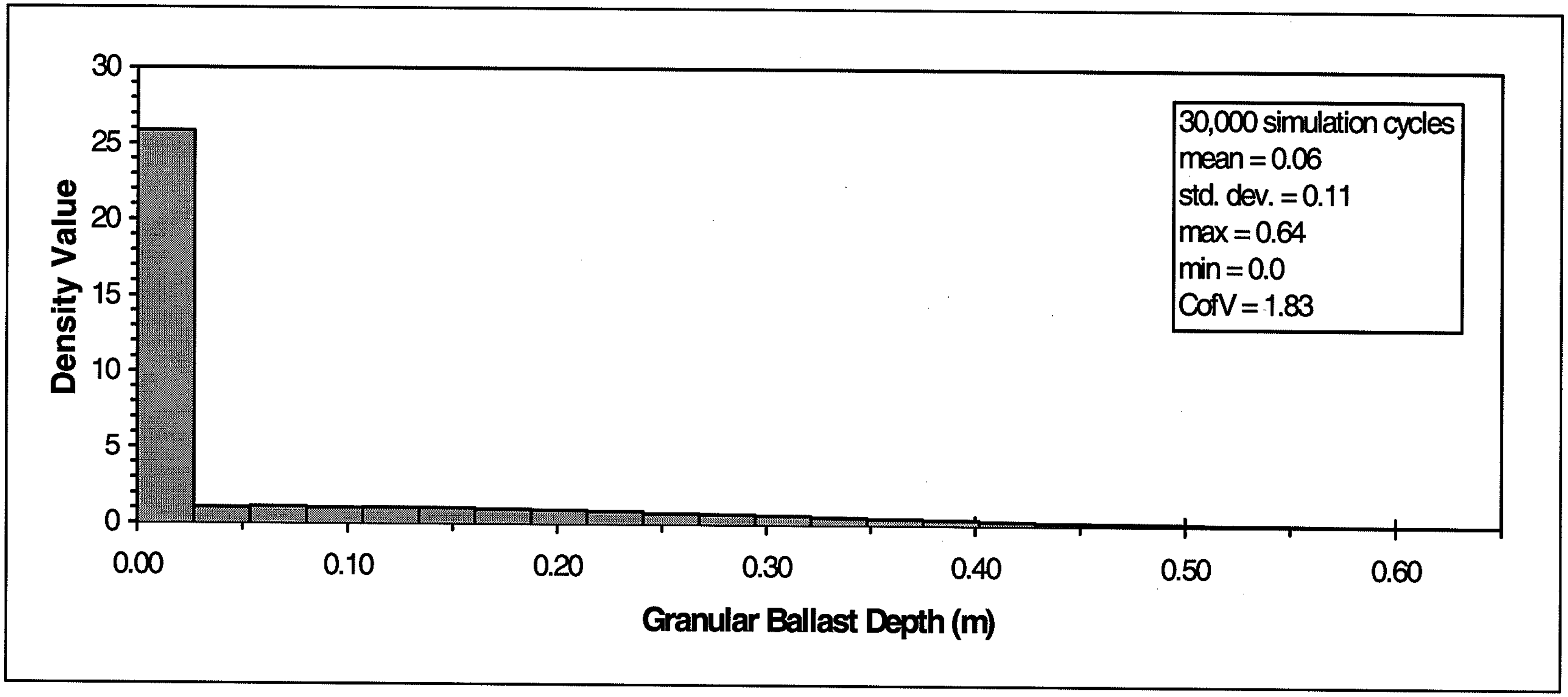


Figure 5-23. Distribution of the depths of granular ballast calculated for the 'BP' aqueduct section using 30,000 simulation cycles. The depths represent the amount of granular ballast required to provide a resisting ratio of 1.0.

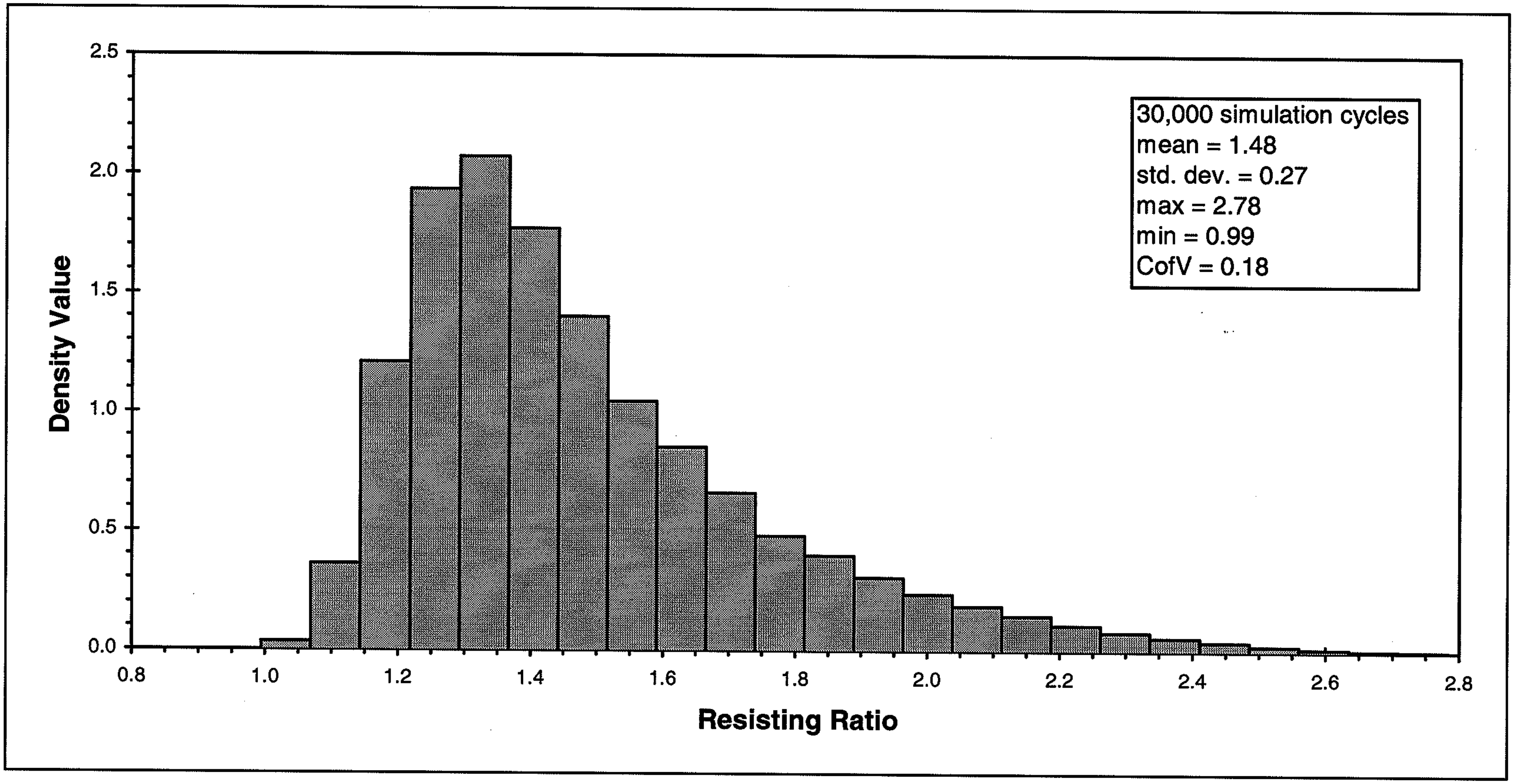


Figure 5-24. Distribution of resisting ratios calculated for the 'BP' aqueduct section including the weight of granular ballast (design depth = 0.65 metres). The probability that the resisting ratio is less than 1.0 is 0.003 percent. The target probability of failure (P_f = 0.011%) corresponds to a resisting ratio of 1.017.

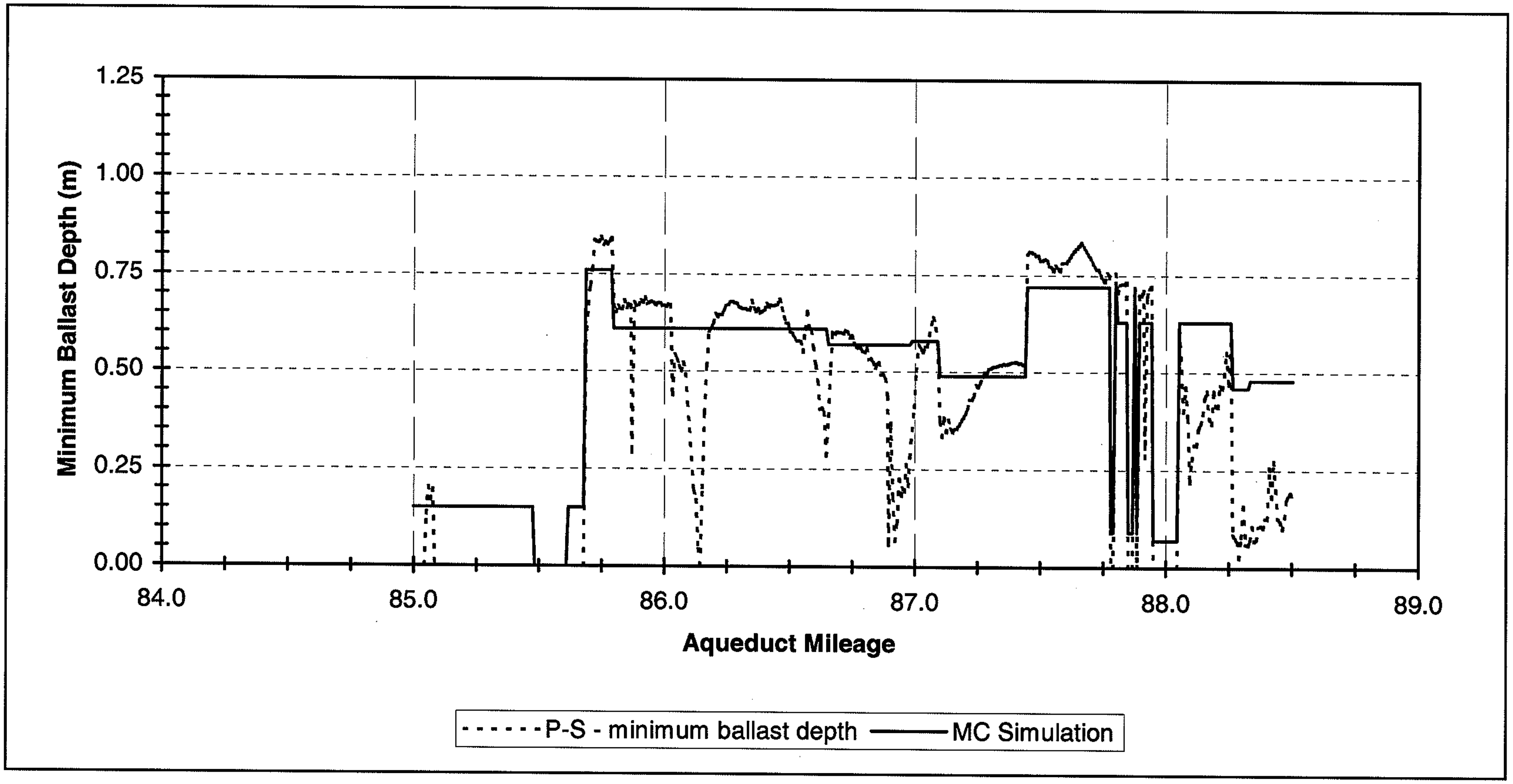


Figure 5-25. Comparison of the profiles of the minimum depths of granular ballast determined using project-specific (P-S) partial safety factors and Monte Carlo (MC) simulation. The depths of ballast are the minimum depths required to provide a resisting ratio of 1.0.

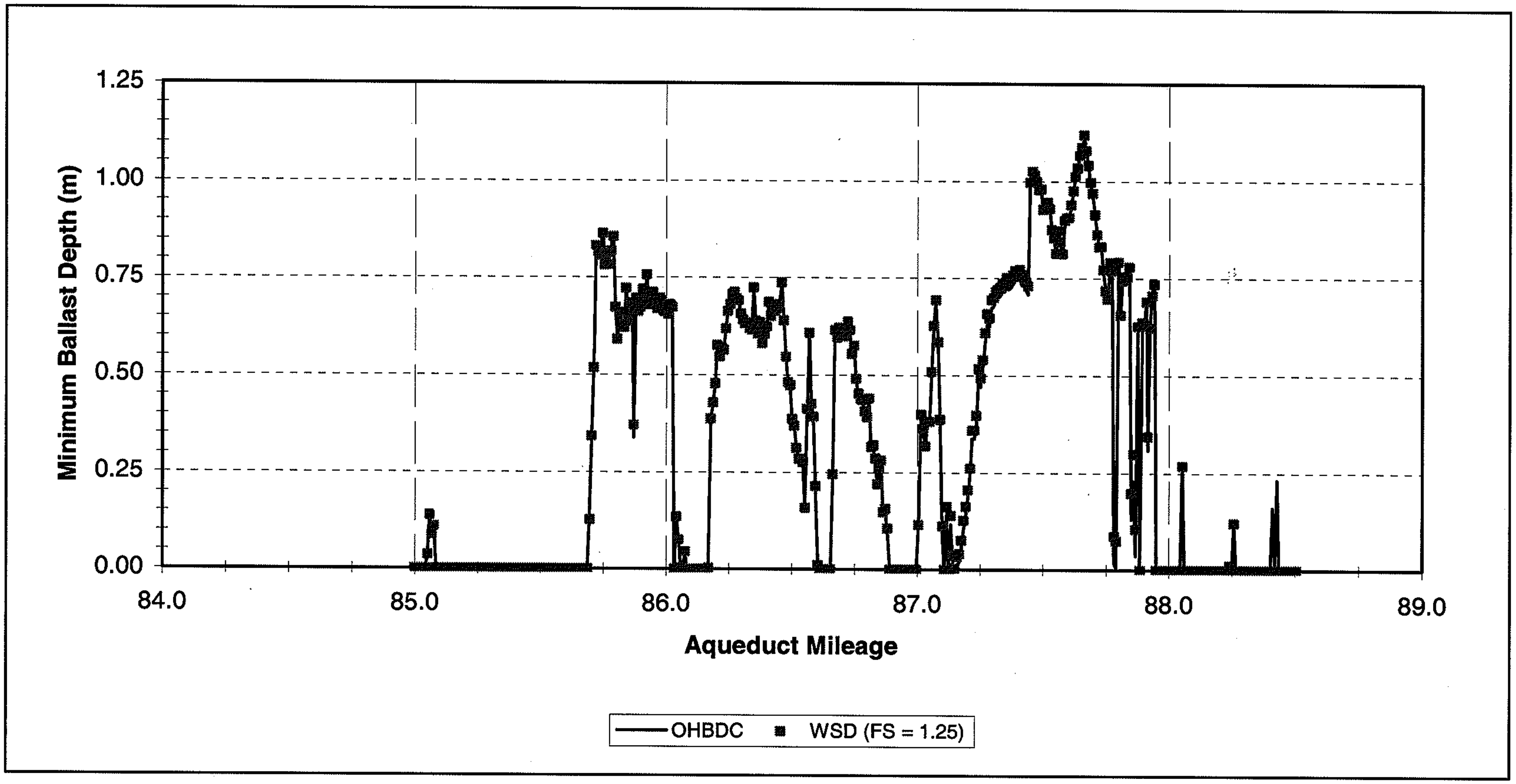


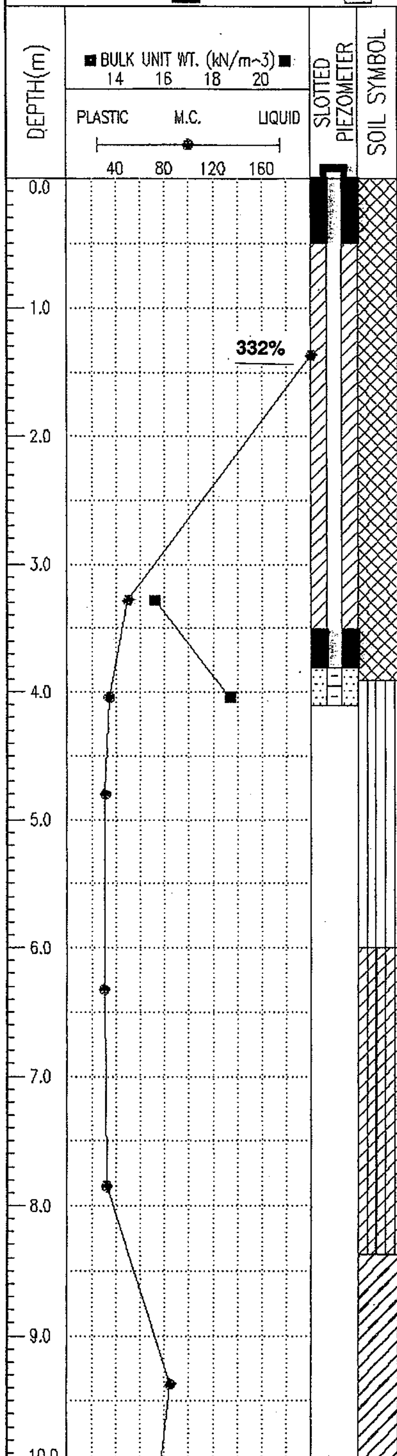
Figure 5-26. Comparison of the minimum depths of granular ballast calculated using the limit state design method using PSFs from the OHBDC (1991) and using the working stress design method with a global factor of safety = 1.25.

APPENDIX 4-1. Test hole logs.

PROJECT: AQUEDUCT BUOYANCY PROGRAM	DRILLED BY: PADDOCK DRILLING	BOREHOLE NO: 1
CLIENT: GORE & STORRIE/CITY OF WINNIPEG	DRILL TYPE: MOBILE B61 C/W 200mmHS AUGER	PROJECT NO: 4895-006-04-04
PROJECT ENGINEER: KMS	LOCATION: MILE 95.000, 3.2m S OF CL	ELEVATION: 323.81 (m)

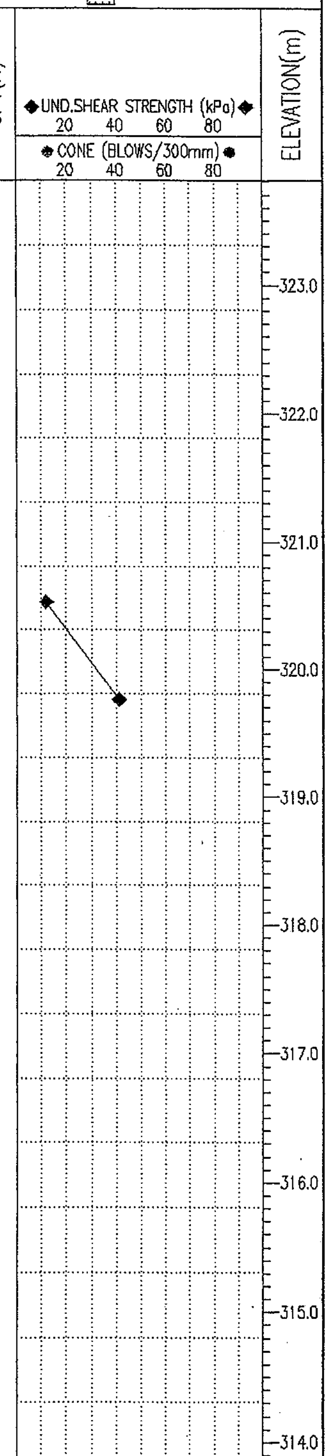
SAMPLE TYPE: GRAB SAMPLE, SHELBY TUBE, SPLIT SPOON, NO RECOVERY, FIELD VANE, WIRE LINE

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND



Soil Description

Depth (m)	Soil Description	Sample Type	Sample No	SPT(N)
0.0	FILL –Organics (Peat) –frozen to 0.76 metres –moist, compressible –clayey, trace of grey silt –brown fibrous	Grab sample	193	
	–dark brown, fibrous/woody –intermixed with grey silt –some fine sand, trace of clay (<5%) –layered, wet, loose –Organics (Peat)	Shelby tube	194	
		Shelby tube	195	
	******* AQUEDUCT INVERT AT 4.1m ********** SILT –clayey, trace of fine sand –very fine grained –13% sand, 59% silt, 28% clay –wet, loose –layered (25mm thick) –grey	Split spoon	196	
	SILT –clayey –with layers of very fine grained sand –brown with red (oxide) stains –wet, loose	Split spoon	197	6
		Split spoon	198	
	CLAY –silty, highly plastic –grey, wet very soft to soft with increasing depth	Split spoon	199	2



UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 25.0 m
	REVIEWED BY: KMS	COMPLETE: 24/02/94
	Fig. No:	Page 1 of 3

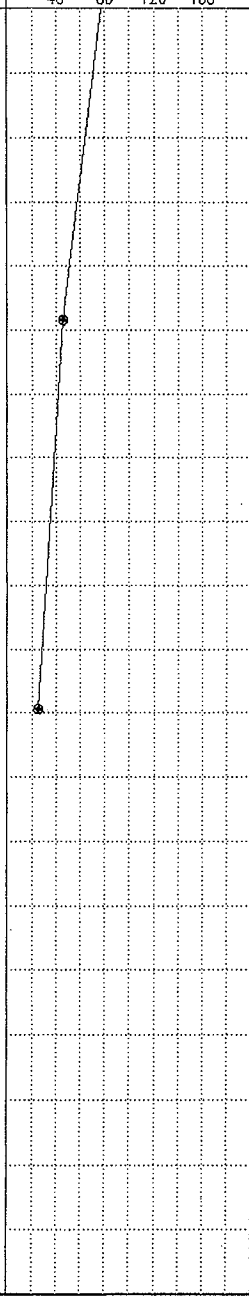
02/05/20 01:01PM (AQUEBUOY)

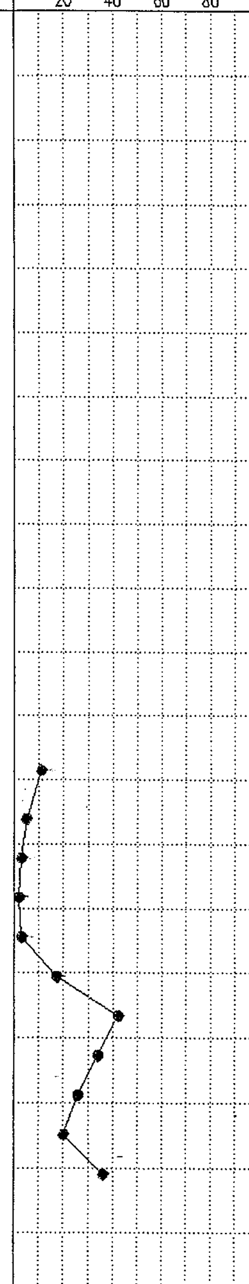
PROJECT: AQUEDUCT BUOYANCY PROGRAM	DRILLED BY: PADDOCK DRILLING	BOREHOLE NO: 1
CLIENT: GORE & STORRIE/CITY OF WINNIPEG	DRILL TYPE: MOBILE B61 C/W 200mmHS AUGER	PROJECT NO: 4895-006-04-04
PROJECT ENGINEER: KMS	LOCATION: MILE 95.000, 3.2m S OF CL	ELEVATION: 323.81 (m)

SAMPLE TYPE: GRAB SAMPLE, SHELBY TUBE, SPLIT SPOON, NO RECOVERY, FIELD VANE, WIRE LINE

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND

DEPTH(m)	BULK UNIT WT. (kN/m^3) 14 16 18 20 / PLASTIC M.C. LIQUID 40 80 120 160	SOIL SYMBOL	Soil Description	SAMPLE TYPE	SAMPLE NO	SPT(N)	UND.SHEAR STRENGTH (kPa) 20 40 60 80 / CONE (BLOWS/300mm) 20 40 60 80	ELEVATION(m)
10.0–10.7		Drill cuttings						
10.7–13.8		Clayey sand	SAND -clayey to 11 metres -trace of silt, grey -fine grained -wet, loose					313.0
12.4				Split spoon	200	5		312.0, 311.0
13.8–15.7		Sand	SAND -medium grained -wet, loose					310.0, 309.0
15.5				Split spoon	201	4		
15.7–19.2		Bentonite	END OF AUGER HOLE AT 15.70 METRES DUTCH CONE DRIVEN TO REFUSAL AT 19.2m SOIL STRATIGRAPHY UNDETERMINED					308.0, 307.0, 306.0, 305.0
19.2–20.0			CONE REFUSAL AT 19.2 METRES NOTES- 3 METRES OF SAND BLOWN UP INTO AUGERS AT 15.7 METRES					304.0





UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 25.0 m
	REVIEWED BY: KMS	COMPLETE: 24/02/94
	Fig. No:	Page 2 of 3

02/05/20 01:02PM (AQUEBUOY)

PROJECT: AQUEDUCT BUOYANCY PROGRAM	DRILLED BY: PADDOCK DRILLING	BOREHOLE NO: 1
CLIENT: GORE & STORRIE/CITY OF WINNIPEG	DRILL TYPE: MOBILE B61 C/W 200mmHS AUGER	PROJECT NO: 4895-006-04-04
PROJECT ENGINEER: KMS	LOCATION: MILE 95.000, 3.2m S OF CL	ELEVATION: 323.81 (m)

SAMPLE TYPE: GRAB SAMPLE, SHELBY TUBE, SPLIT SPOON, NO RECOVERY, FIELD VANE, WIRE LINE

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND

DEPTH(m)	BULK UNIT WT. (kN/m~3) 14 16 18 20 / PLASTIC M.C. LIQUID 40 80 120 160		SOIL SYMBOL	Soil Description	SAMPLE TYPE	SAMPLE NO	SPT(N)	UND.SHEAR STRENGTH (kPa) 20 40 60 80 / CONE (BLOWS/300mm) 20 40 60 80	ELEVATION(m)
20.0				- SEEPAGE DURING DRILLING - STANDPIPE PIEZOMETER P1 INSTALLED TOP OF PIPE AT ELEV. 325.01m					
									303.0
21.0									
									302.0
22.0									
									301.0
23.0									
									300.0
24.0									
									299.0
25.0									
									298.0
26.0									
									297.0
27.0									
									296.0
28.0									
									295.0
29.0									
30.0									294.0

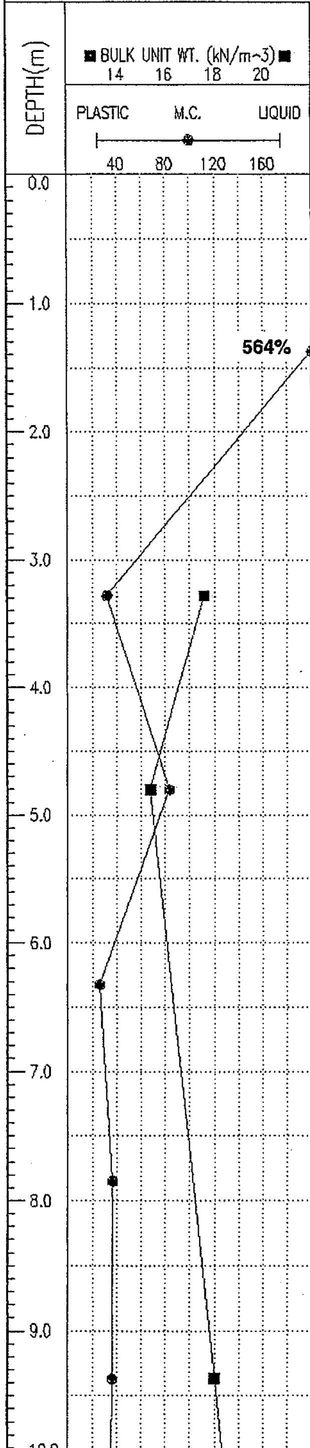
UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 25.0 m
	REVIEWED BY: KMS	COMPLETE: 24/02/94
	Fig. No:	Page 3 of 3

02/05/20 01:02PM (AQUEBUOY)

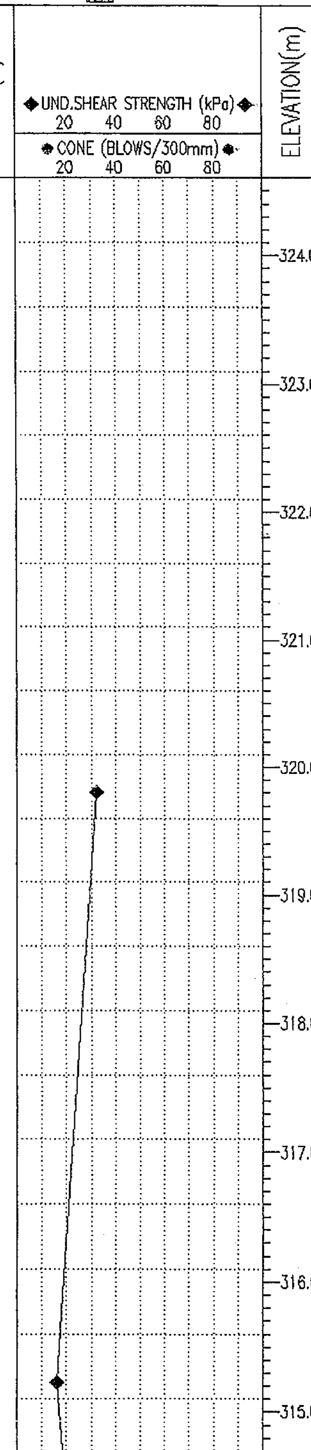
PROJECT: AQUEDUCT BUOYANCY PROGRAM	DRILLED BY: PADDOCK DRILLING	BOREHOLE NO: 8
CLIENT: GORE & STORRIE/CITY OF WINNIPEG	DRILL TYPE: MOBILE B61 C/W 200mmHS AUGER	PROJECT NO: 4895-006-04-04
PROJECT ENGINEER: KMS	LOCATION: MILE 91.400, 32.6m S OF CL	ELEVATION: 324.6 (m)

SAMPLE TYPE: GRAB SAMPLE, SHELBY TUBE, SPLIT SPOON, NO RECOVERY, FIELD VANE, WIRE LINE

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND



Depth (m)	Soil Description	Sample Type	Sample No
0.0–2.2	ORGANICS (PEAT) -frozen to 0.45 metres -fibrous -brown -wet -compressible	Grab sample	138
2.2–3.9	SILT -light grey, with fine sand, wet, loose	Shelby tube	139
3.9–5.4	CLAY -grey, silty, high plasticity -trace of organics (rootlets), wet -firm to soft	Shelby tube	140
5.4–10.0	CLAY ***AQUEDUCT INVERT AT 5.58m ****** -grey, silty, intermediate plasticity -trace of sand and gravel (10-12mm dia typical) -wet, soft	Grab sample	141
		Grab sample	142
		Shelby tube	143



UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 15.9 m
	REVIEWED BY: KMS	COMPLETE: 21/02/94
	Fig. No:	Page 1 of 2

02/05/20 01:02PM (AQUEBUOY)

PROJECT: AQUEDUCT BUOYANCY PROGRAM	DRILLED BY: PADDOCK DRILLING	BOREHOLE NO: 8
CLIENT: GORE & STORRIE/CITY OF WINNIPEG	DRILL TYPE: MOBILE B61 C/W 200mmHS AUGER	PROJECT NO: 4895-006-04-04
PROJECT ENGINEER: KMS	LOCATION: MILE 91.400, 32.6m S OF CL	ELEVATION: 324.6 (m)

SAMPLE TYPE: GRAB SAMPLE, SHELBY TUBE, SPLIT SPOON, NO RECOVERY, FIELD VANE, WIRE LINE

BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND

Columns: DEPTH(m) | ■ BULK UNIT WT. (kN/m^3) ■ 14 16 18 20; PLASTIC M.C. LIQUID 40 80 120 160 | INSTRUMENTATION DATA | SOIL SYMBOL | Soil Description | SAMPLE TYPE | SAMPLE NO | SPT(N) | ◆ UND.SHEAR STRENGTH (kPa) ◆ 20 40 60 80; ● CONE (BLOWS/300mm) ● 20 40 60 80 | ELEVATION(m)

Depth scale: 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0

Elevation scale: 314.0, 313.0, 312.0, 311.0, 310.0, 309.0, 308.0, 307.0, 306.0, 305.0

Soil Description

CLAY
- grey
- intermediate plasticity
- trace of sand and gravel (10-12mm typical)
- wet
- soft

Sample No: 144, 145

AUGER REFUSAL AT 15.85 METRES

NOTES - SEEPAGE DURING DRILLING
- NO PIEZOMETER INSTALLED

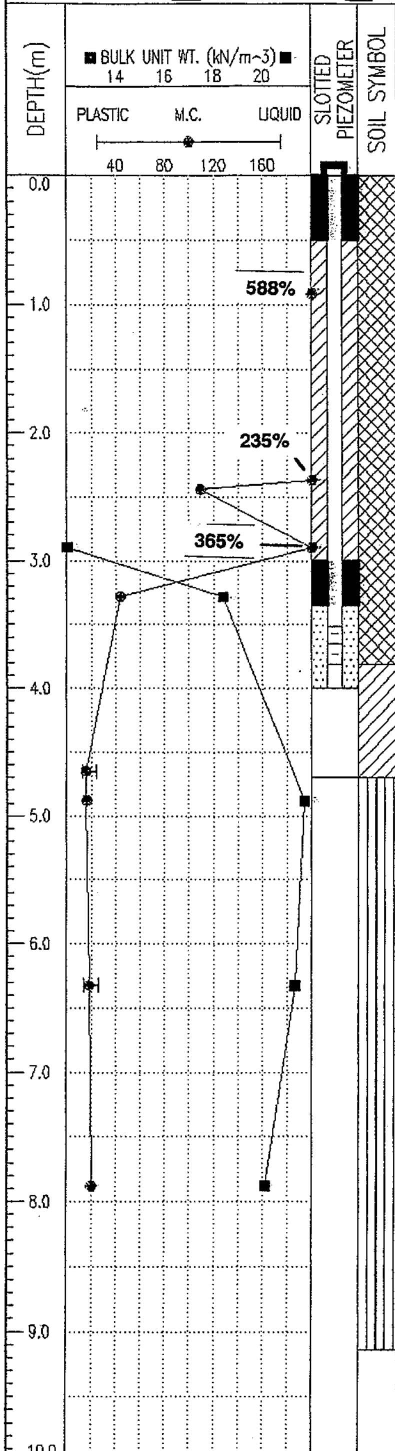
UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 15.9 m
	REVIEWED BY: KMS	COMPLETE: 21/02/94
	Fig. No:	Page 2 of 2

02/05/20 01:02PM (AQUEBUOY)

PROJECT: AQUEDUCT BUOYANCY PROGRAM	DRILLED BY: PADDOCK DRILLING	BOREHOLE NO: 14
CLIENT: GORE & STORRIE/CITY OF WINNIPEG	DRILL TYPE: MOBILE B61 C/W 200mmHS AUGER	PROJECT NO: 4895-006-04-04
PROJECT ENGINEER: KMS	LOCATION: MILE 87.071, 2.9m S OF CL	ELEVATION: 320.54 (m)

SAMPLE TYPE: GRAB SAMPLE, SHELBY TUBE, SPLIT SPOON, NO RECOVERY, FIELD VANE, WIRE LINE

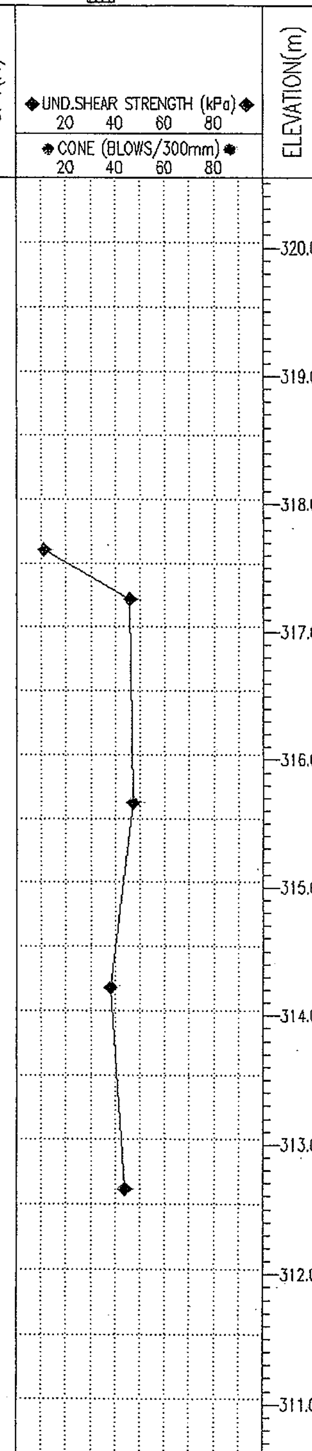
BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND



Soil Description

Depth	Soil Description	Sample No
0.0	FILL –Organics (Peat) –fibrous, brown –wet, very compressible	96
	–Organics (Peat) –trace of fine grey silty sand, brown –fibrous, wet, very compressible	97, 98
	–Clay –with organics (layered with peat) –silty, some sand inclusions –grey, wet, soft ******AQUEDUCT INVERT AT 3.41 METRES******	99, 100
	CLAY –silty –trace of gravel, low plasticity –wet, firm	
	SILT –clayey –brown to grey –trace of sand and gravel –moist to wet –compact	101, 102, 103
9.14	AUGER REFUSAL AT 9.14 METRES NOTES – SEEPAGE DURING DRILLING – STANDPIPE PIEZOMETER P14 INSTALLED	

SAMPLE TYPE | SAMPLE NO | SPT(N)



UMA Engineering Ltd. Winnipeg, Manitoba	LOGGED BY: DY	COMPLETION DEPTH: 9.1 m
	REVIEWED BY: KMS	COMPLETE: 17/02/94
	Fig. No:	Page 1 of 1

02/05/20 01:02PM (AQUEBUOY)

APPENDIX 5-1. Spreadsheet model used with partial safety factors from OHBDC (1991).

SHOAL LAKE AQUEDUCT BUOYANCY ASSESSMENT - Boggy River Stretch (Mile 85.0 to 88.5) - Partial Safety Factor Method Using OHBDC (1991) Load and Resistance Factors

General Buoyancy Equation: [$\phi_{backfill}$ (backfill weight) + ϕ_{shear} (backfill shear resistance) + $\phi_{concrete}$ (weight of aqueduct) + $\phi_{ballast}$ (granular ballast weight)] $\geq \alpha$ (buoyant force)

Backfill Notes
- measured or predicted unit weight
- submerged unit weights
- bulk unit weights (above crown only)

Backfill Shear Resistance:
Depth of Compacted Backfill Above Invert
- 0.0 = peat backfill (no shear resistance)
- 1.0 = clay layer (m) (depth contributing to shear resistance)

Concrete Unit Weight and Resistance Factor
- $\gamma_{Concrete}$ = 22.50 (kN/m³)
- Resistance Factor, ϕconcrete = 0.90

Backfill Resistance Factors
- Submerged Backfill, ϕsub. = 0.80
- Non-Submerged Crown Backfill, ϕbulk = 0.80
- Backfill Shear, ϕshear = 0.50
- Undrained Shear Strength = 12.0 (kPa)

Buoyancy Load Parameters
- γ_{water} = 9.81 (kN/m³)
- Buoyancy Load Factor, α = 1.05

Granular Ballast Parameters
- $\gamma_{Ballast\ bulk}$ = 14.40 (kN/m³)
- Resistance Factor, ϕbulk = 0.80 Non-Submerged ballast
- $\gamma_{Ballast\ Saturated}$ = 18.00 (kN/m³)
- Resistance Factor, ϕsub = 0.80 Submerged Ballast

Aqueduct Mileage	Aqueduct Segment Section Type	Aqueduct Segment No.	Aqueduct Survey Crown Elevation (m)	Invert Elevations Inside Aqueduct (m)	Invert Elevations Foundation Side (m)	Aqueduct Dimensions Inside Height (m)	Outside Height (m)	Area Below Haunch Grade (m²)	Maximum Exterior Width (m)	Invert Thickness (mm)	Crown Thickness (mm)	Concrete Area Invert (m²)	Concrete Area Arch (m²)	Concrete Area Total (m²)	Concrete Weight (kN/m)	Backfill Properties Average Backfill Elevation (m)	Depth on Centerline (m)	Saturated Unit Weight Crown (kN/m³)	Saturated Unit Weight Arch Leg (kN/m³)	Backfill Weight (kN/m)	Backfill Shear Resistance (kN/m)	Total Resistance R (kN/m)	Aqueduct Cross-section Area Inside (m²)	Aqueduct Cross-section Area Total (m²)	Buoyant Force B (kN/m)	Resisting Ratio R / B	Force to Resist (R / B<1) (kN/m)	Granular Ballast Minimum Depth Required (m)	Repair Length (m)	Design Depth (m)	Additional Resistance (kN/m)	New Total Resistance R (kN/m)	New Resisting Ratio R / B
84.999	BO	1236	318.52	316.11	315.96	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.34	0.823	12.09	12.09	36.55	0	75.87	4.64	6.58	67.76	1.12	0.0	0.00	0.00	0.00	0.00	75.87	1.12
85.008	BO	1235	318.52	316.12	315.96	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.34	0.815	12.29	12.29	37.39	0	76.72	4.64	6.58	67.76	1.13	0.0	0.00	0.00	0.20	8.43	85.15	1.26
85.016	BO	1234	318.53	316.13	315.97	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.26	0.726	12.49	12.49	35.23	0	74.56	4.64	6.58	67.76	1.10	0.0	0.00	0.00	0.20	8.43	82.98	1.22
85.025	BO	1233	318.54	316.13	315.98	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.38	0.846	12.49	12.49	39.68	0	79.01	4.64	6.58	67.76	1.17	0.0	0.00	0.00	0.20	8.43	87.44	1.29
85.033	BO	1232	318.54	316.13	315.98	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.34	0.805	12.50	12.50	38.21	0	77.54	4.64	6.58	67.76	1.14	0.0	0.00	0.00	0.20	8.43	85.97	1.27
85.042	BO	1231	318.53	316.13	315.98	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.31	0.781	12.51	12.51	37.32	0	76.65	4.64	6.58	67.76	1.13	0.0	0.00	0.00	0.20	8.43	85.08	1.26
85.050	BO	1230	318.54	316.14	315.98	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.03	0.490	12.51	12.51	26.74	0	66.06	4.64	6.58	67.76	0.97	1.7	0.04	13.67	0.20	8.43	74.49	1.10
85.059	BO	1229	318.54	316.14	315.98	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	318.91	0.373	12.51	12.51	22.47	0	61.80	4.64	6.58	67.76	0.91	6.0	0.14	13.71	0.20	8.43	70.23	1.04
85.067	BO	1228	318.54	316.14	315.99	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	318.97	0.430	12.52	12.52	24.55	0	63.87	4.64	6.58	67.76	0.94	3.9	0.09	13.73	0.20	8.43	72.30	1.07
85.076	BO	1227	318.54	316.13	315.98	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	318.90	0.361	12.90	12.90	23.65	0	62.97	4.64	6.58	67.76	0.93	4.8	0.11	13.72	0.20	8.43	71.40	1.05
85.084	BO	1226	318.52	316.12	315.97	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.19	0.663	13.28	13.28	37.05	0	76.38	4.64	6.58	67.76	1.13	0.0	0.00	0.00	0.20	8.43	84.80	1.25
85.093	BO	1225	318.54	316.14	315.99	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.25	0.708	13.65	13.65	40.81	0	80.13	4.64	6.58	67.76	1.18	0.0	0.00	0.00	0.00	0.00	80.13	1.18
85.101	BO	1224	318.54	316.14	315.99	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.46	0.916	14.03	14.03	51.37	0	90.69	4.64	6.58	67.76	1.34	0.0	0.00	0.00	0.00	0.00	90.69	1.34
85.110	BO	1223	318.54	316.14	315.98	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.42	0.880	14.41	14.41	52.04	0	91.37	4.64	6.58	67.76	1.35	0.0	0.00	0.00	0.00	0.00	91.37	1.35
85.119	BO	1222	318.54	316.14	315.99	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.32	0.776	14.22	14.22	46.67	0	86.00	4.64	6.58	67.76	1.27	0.0	0.00	0.00	0.00	0.00	86.00	1.27
85.127	BO	1221	318.55	316.15	315.99	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.44	0.889	14.04	14.04	50.28	0	89.61	4.64	6.58	67.76	1.32	0.0	0.00	0.00	0.00	0.00	89.61	1.32
85.136	BO	1220	318.56	316.16	316.00	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.50	0.942	13.85	13.85	51.31	0	90.63	4.64	6.58	67.76	1.34	0.0	0.00	0.00	0.00	0.00	90.63	1.34
85.138	BO	1219	318.57	316.16	316.01	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.43	0.861	13.79	13.79	47.67	0	86.99	4.64	6.58	67.76	1.28	0.0	0.00	0.00	0.00	0.00	86.99	1.28
85.147	BO	1218	318.58	316.17	316.02	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.25	0.673	14.02	14.02	41.28	0	80.61	4.64	6.58	67.76	1.19	0.0	0.00	0.00	0.00	0.00	80.61	1.19
85.156	BO	1217	318.58	316.18	316.03	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.60	1.019	14.25	14.25	56.91	0	96.23	4.64	6.58	67.76	1.42	0.0	0.00	0.00	0.00	0.00	96.23	1.42
85.164	BO	1216	318.58	316.18	316.02	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.35	0.771	14.34	14.34	47.11	0	86.44	4.64	6.58	67.76	1.28	0.0	0.00	0.00	0.00	0.00	86.44	1.28
85.173	BO	1215	318.59	316.19	316.03	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.26	0.671	14.43	14.43	43.35	0	82.67	4.64	6.58	67.76	1.22	0.0	0.00	0.00	0.00	0.00	82.67	1.22
85.181	BO	1214	318.59	316.19	316.03	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.42	0.834	14.52	14.52	50.79	0	90.11	4.64	6.58	67.76	1.33	0.0	0.00	0.00	0.00	0.00	90.11	1.33
85.190	BO	1213	318.59	316.19	316.04	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.54	0.949	14.61	14.61	56.18	0	95.51	4.64	6.58	67.76	1.41	0.0	0.00	0.00	0.00	0.00	95.51	1.41
85.198	BO	1212	318.60	316.19	316.04	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.64	1.037	14.70	14.70	60.56	0	99.88	4.64	6.58	67.76	1.47	0.0	0.00	0.00	0.00	0.00	99.88	1.47
85.207	BO	1211	318.59	316.19	316.03	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.46	0.868	14.80	14.80	53.77	0	93.10	4.64	6.58	67.76	1.37	0.0	0.00	0.00	0.00	0.00	93.10	1.37
85.215	BO	1210	318.60	316.20	316.05	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.60	0.998	14.89	14.89	59.96	0	99.29	4.64	6.58	67.76	1.47	0.0	0.00	0.00	0.00	0.00	99.29	1.47
85.224	BO	1209	318.62	316.21	316.06	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.67	1.048	14.98	14.98	62.75	0	102.08	4.64	6.58	67.76	1.51	0.0	0.00	0.00	0.00	0.00	102.08	1.51
85.232	BO	1208	318.61	316.21	316.06	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.86	1.243	15.07	15.07	71.88	0	111.21	4.64	6.58	67.76	1.64	0.0	0.00	0.00	0.00	0.00	111.21	1.64
85.241	BO	1207	318.62	316.22	316.07	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.67	1.048	15.16	15.16	63.86	0	103.18	4.64	6.58	67.76	1.52	0.0	0.00	0.00	0.00	0.00	103.18	1.52
85.249	BO	1206	318.63	316.22	316.07	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.88	1.255	15.25	15.25	73.67	0	113.00	4.64	6.58	67.76	1.67	0.0	0.00	0.00	0.00	0.00	113.00	1.67
85.258	BO	1205	318.62	316.22	316.06	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.90	1.274	15.07	15.07	73.30	0	112.62	4.64	6.58	67.76	1.66	0.0	0.00	0.00	0.00	0.00	112.62	1.66
85.266	BO	1204	318.65	316.25	316.10	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.78	1.123	14.89	14.89	65.40	0	104.72	4.64	6.58	67.76	1.55	0.0	0.00	0.00	0.00	0.00	104.72	1.55
85.275	BO	1203	318.64	316.24	316.09	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.48	0.838	14.70	14.70	51.94	0	91.27	4.64	6.58	67.76	1.35	0.0	0.00	0.00	0.00	0.00	91.27	1.35
85.284	BO	1202	318.65	316.25	316.10	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.35	0.700	14.52	14.52	45.04	0	84.36	4.64	6.58	67.76	1.25	0.0	0.00	0.00	0.00	0.00	84.36	1.25
85.292	BO	1201	318.65	316.25	316.10	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	320.10	1.445	14.34	14.34	75.37	0	114.70	4.64	6.58	67.76	1.69	0.0	0.00	0.00	0.00	0.00	114.70	1.69
85.301	BO	1200	318.66	316.26	316.10	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.94	1.282	14.15	14.15	67.25	0	106.58	4.64	6.58	67.76	1.57	0.0	0.00	0.00	0.00	0.00	106.58	1.57
85.309	BO	1199	318.67	316.26	316.11	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.94	1.278	13.97	13.97	65.73	0	105.06	4.64	6.58	67.76	1.55	0.0	0.00	0.00	0.00	0.00	105.06	1.55
85.318	BO	1198	318.67	316.26	316.11	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.69	1.022	13.78	13.78	54.17	0	93.49	4.64	6.58	67.76	1.38	0.0	0.00	0.00	0.00	0.00	93.49	1.38
85.327	BO	1197	318.68	316.27	316.12	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.42	0.747	13.60	13.60	42.01	0	81.33	4.64	6.58	67.76	1.20	0.0	0.00	0.00	0.00	0.00	81.33	1.20
85.335	BO	1196	318.68	316.27	316.12	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.77	1.092	13.41	13.41	54.60	0	93.92	4.64	6.58	67.76	1.39	0.0	0.00	0.00	0.00	0.00	93.92	1.39
85.344	BO	1195	318.69	316.28	316.13	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	320.08	1.390	13.23	13.23	64.92	0	104.24	4.64	6.58	67.76	1.54	0.0	0.00	0.00	0.00	0.00	104.24	1.54
85.352	BO	1194	318.67	316.27	316.12	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.96	1.288	13.33	13.33	61.67	0	101.00	4.64	6.58	67.76	1.49	0.0	0.00	0.00	0.00	0.00	101.00	1.49
85.361	BO	1193	318.68	316.28	316.13	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	320.08	1.390	13.43	13.43	66.42	0	105.75	4.64	6.58	67.76	1.56	0.0	0.00	0.00	0.00	0.00	105.75	1.56
85.369	BO	1192	318.68	316.28	316.13	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.87	1.185	13.53	13.53	59.00	0	98.32	4.64	6.58	67.76	1.45	0.0	0.00	0.00	0.00	0.00	98.32	1.45
85.378	BO	1191	318.69	316.29	316.14	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.88	1.193	13.63	13.63	59.98	0	99.31	4.64	6.58	67.76	1.47	0.0	0.00	0.00	0.00	0.00	99.31	1.47
85.386	BO	1190	318.70	316.30	316.14	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.66	0.957	13.73	13.73	51.21	0	90.54	4.64	6.58	67.76	1.34	0.0	0.00	0.00	0.00	0.00	90.54	1.34
85.395	BO	1189	318.70	316.29	316.14	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.84	1.142	13.83	13.83	59.27	0	98.59	4.64	6.58	67.76	1.46	0.0	0.00	0.00	0.00	0.00	98.59	1.46
85.404	BO	1188	318.70	316.30	316.14	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.64	0.936	13.93	13.93	51.57	0	90.90	4.64	6.58	67.76	1.34	0.0	0.00	0.00	0.00	0.00	90.90	1.34
85.412	BO	1187	318.70	316.30	316.15	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.59	0.882	14.03	14.03	49.95	0	89.28	4.64	6.58	67.76	1.32	0.0	0.00	0.00	0.00	0.00	89.28	1.32
85.421	BO	1186	318.71	316.31	316.16	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.66	0.950	14.13	14.13	53.31	0	92.63	4.64	6.58	67.76	1.37	0.0	0.00	0.00	0.00	0.00	92.63	1.37
85.429	BO	1185	318.71	316.31	316.15	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.80	1.089	14.23	14.23	59.70	0	99.02	4.64	6.58	67.76	1.46	0.0	0.00	0.00	0.00	0.00	99.02	1.46
85.438	BO	1184	318.71	316.31	316.16	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.69	0.974	14.33	14.33	55.52	0	94.85	4.64	6.58	67.76	1.40	0.0	0.00	0.00	0.00	0.00	94.85	1.40
85.446	BO	1183	318.73	316.33	316.17	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.64	0.909	14.11	14.11	51.51	0	90.83	4.64	6.58	67.76	1.34	0.0	0.00	0.00	0.00	0.00	90.83	1.34
85.455	BO	1182	318.72	316.32	316.16	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.59	0.870	13.88	13.88	48.59	0	87.91	4.64	6.58	67.76	1.30	0.0	0.00	0.00	0.00	0.00	87.91	1.30
85.463	BO	1181	318.73	316.33	316.18	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.61	0.880	13.65	13.65	47.68	0	87.01	4.64	6.58	67.76	1.28	0.0	0.00	0.00	0.00	0.00	87.01	1.28
85.472	BO	1180	318.71	316.31	316.16	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.56	0.841	13.43	13.43	44.83	0	84.15	4.64	6.58	67.76	1.24	0.0	0.00	0.00	0.00	0.00	84.15	1.24
85.475	BO	1179	318.71	316.31	316.15	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.51	0.796	13.35	13.35	42.64	0	81.97	4.64	6.58	67.76	1.21	0.0	0.00	0.00	0.00	0.00	81.97	1.21
85.483	B Piled	1178	318.70	316.30	315.94	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.40	0.696	13.13	13.13	37.40	0	100.58	4.64	7.76	79.89	1.26	0.0	0.00	0.00	0.00	0.00	100.58	1.26
85.492	B Piled	1177	318.70	316.30	315.94	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.33	0.632	12.90	12.90	33.71	0	96.89	4.64	7.76	79.89	1.21	0.0	0.00	0.00	0.00	0.00	96.89	1.21
85.500	B Piled	1176	318.73	316.33	315.97	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.38	0.654	12.67	12.67	33.53	0	96.71	4.64	7.76	79.89	1.21	0.0	0.00	0.00	0.00	0.00	96.71	1.21
85.509	B Piled	1175	318.73	316.33	315.98	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.45	0.717	12.45	12.45	35.01	0	98.19	4.64	7.76	79.89	1.23	0.0	0.00	0.00	0.00	0.00	98.19	1.23
85.517	B Piled	1174	318.73	316.33	315.97	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.58	0.848	12.22	12.22	38.96	0	102.14	4.64	7.76	79.89	1.28	0.0	0.00	0.00	0.00	0.00	102.14	1.28
85.526	B Piled	1173	318.73	316.32	315.97	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.76	1.030	12.00	12.00	44.73	0	107.91	4.64	7.76	79.89	1.35	0.0	0.00	0.00	0.00	0.00	107.91	1.35
85.534	B Piled	1172	318.72	316.32	315.96	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.80	1.080	12.15	12.15	47.57	0	110.75	4.64	7.76	79.89	1.39	0.0	0.00	0.00	0.00	0.00	110.75	1.39
85.543	B Piled	1171	318.72	316.31	315.96	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.55	0.829	12.31	12.31	38.67	0	101.85	4.64	7.76	79.89	1.27	0.0	0.00	0.00	0.00	0.00	101.85	1.27
85.551	B Piled	1170	318.72	316.31	315.96	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.68	0.969	12.46	12.46	45.01	0	108.19	4.64	7.76	79.89	1.35	0.0	0.00	0.00	0.00	0.00	108.19	1.35
85.560	B Piled	1169	318.74	316.33	315.98	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.61	0.873	12.62	12.62	42.08	0	105.26	4.64	7.76	79.89	1.32	0.0	0.00	0.00	0.00	0.00	105.26	1.32
85.568	B Piled	1168	318.73	316.33	315.97	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.59	0.856	12.78	12.78	42.22	0	105.40	4.64	7.76	79.89	1.32	0.0	0.00	0.00	0.00	0.00	105.40	1.32
85.577	B Piled	1167	318.75	316.35	315.99	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.79	1.038	12.93	12.93	50.46	0	113.64	4.64	7.76	79.89	1.42	0.0	0.00	0.00	0.00	0.00	113.64	1.42
85.586	B Piled	1166	318.75	316.35	315.99	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.60	0.844	13.09	13.09	43.32	0	106.50	4.64	7.76	79.89	1.33	0.0	0.00	0.00	0.00	0.00	106.50	1.33
85.594	B Piled	1165	318.75	316.35	315.99	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.75	0.995	13.24	13.24	50.48	0	113.66	4.64	7.76	79.89	1.42	0.0	0.00	0.00	0.00	0.00	113.66	1.42
85.603	B Piled	1164	318.73	316.33	315.98	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.39	0.655	13.40	13.40	36.97	0	100.15	4.64	7.76	79.89	1.25	0.0	0.00	0.00	0.00	0.00	100.15	1.25
85.608	B Piled	1163	318.77	316.37	316.01	2.251	2.759	0.000	3.962	356	152	1.78	1.34	3.12	63.18	319.84	1.068	13.50	13.50	55.07	0	118.25	4.64	7.76	79.89	1.48	0.0	0.00	0.00	0.00	0.00	118.25	1.48
85.617	BO	1162	318.79	316.38	316.23	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.77	0.985	13.66	13.66	51.87	0	91.20	4.64	6.58	67.76	1.35	0.0	0.00	0.00	0.00	0.00	91.20	1.35
85.625	BO	1161	318.79	316.39	316.24	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.79	0.994	19.02	19.02	85.24	0	124.57	4.64	6.58	67.76	1.84	0.0	0.00	0.00	0.00	0.00	124.57	1.84
85.634	BO	1160	318.78	316.38	316.23	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.52	0.738	20.36	20.36	78.30	0	117.63	4.64	6.58	67.76	1.74	0.0	0.00	0.00	0.00	0.00	117.63	1.74
85.643	BO	1159	318.80	316.40	316.24	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.62	0.817	21.70	21.70	90.57	0	129.90	4.64	6.58	67.76	1.92	0.0	0.00	0.00	0.00	0.00	129.90	1.92
85.652	BO	1158	318.83	316.42	316.27	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.65	0.821	20.47	20.47	83.89	0	123.21	4.64	6.58	67.76	1.82	0.0	0.00	0.00	0.00	0.00	123.21	1.82
85.660	BO	1157	318.83	316.42	316.27	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.56	0.735	19.32	19.32	72.43	0	111.75	4.64	6.58	67.76	1.65	0.0	0.00	0.00	0.00	0.00	111.75	1.65
85.669	BO	1156	318.83	316.43	316.27	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.40	0.575	18.17	18.17	57.76	0	97.09	4.64	6.58	67.76	1.43	0.0	0.00	0.00	0.10	4.21	101.30	1.50
85.677	BO	1155	318.83	316.43	316.27	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.45	0.620	17.01	17.01	54.23	0	93.55	4.64	6.58	67.76	1.38	0.0	0.00	0.00	0.35	14.75	108.30	1.60
85.686	BO	1154	318.84	316.44	316.28	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.74	0.898	15.86	15.86	35.54	0	74.87	4.64	6.58	67.76	1.10	0.0	0.00	0.00	0.60	14.38	89.25	1.32
85.694	BO	1153	318.85	316.44	316.29	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.50	0.653	14.69	14.69	25.21	0	64.54	4.64	6.58	67.76	0.95	3.2	0.12	13.82	0.90	21.57	86.11	1.27
85.703	BO	1152	318.84	316.44	316.29	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.57	0.728	13.54	13.54	20.07	0	59.39	4.64	6.58	67.76	0.88	8.4	0.35	13.70	0.90	21.57	80.96	1.19

SHOAL LAKE AQUEDUCT BUOYANCY ASSESSMENT - Boggy River Stretch (Mile 85.0 to 88.5) - Partial Safety Factor Method Using OHBDC (1991) Load and Resistance Factors

General Buoyancy Equation: [$\phi_{backfill}$ (backfill weight) + ϕ_{shear} (backfill shear resistance) + $\phi_{concrete}$ (weight of aqueduct) + $\phi_{ballast}$ (granular ballast weight)] $\geq \alpha$ (buoyant force)

Backfill Notes
- measured or predicted unit weight
- submerged unit weights
- bulk unit weights (above crown only)

Backfill Shear Resistance:
Depth of Compacted Backfill Above Invert
- 0.0 = peat backfill (no shear resistance)
- 1.0 = clay layer (m) (depth contributing to shear resistance)

Concrete Unit Weight and Resistance Factor
- $\gamma_{Concrete}$ = 22.50 (kN/m³)
- Resistance Factor, ϕconcrete = 0.90

Backfill Resistance Factors
- Submerged Backfill, ϕsub. = 0.80
- Non-Submerged Crown Backfill, ϕbulk = 0.80
- Backfill Shear, ϕshear = 0.50
- Undrained Shear Strength = 12.0 (kPa)

Buoyancy Load Parameters
- γ_{water} = 9.81 (kN/m³)
- Buoyancy Load Factor, α = 1.05

Granular Ballast Parameters
- $\gamma_{Ballast\ bulk}$ = 14.40 (kN/m³)
- Resistance Factor, ϕbulk = 0.80 Non-Submerged ballast
- $\gamma_{Ballast\ Saturated}$ = 13.00 (kN/m³)
- Resistance Factor, ϕsub = 0.80 Submerged Ballast

Aqueduct Mileage	Aqueduct Segment Section Type	Aqueduct Segment No.	Aqueduct Survey Crown Elevation (m)	Invert Elevations Inside Aqueduct (m)	Invert Elevations Foundation Side (m)	Aqueduct Dimensions Inside Height (m)	Outside Height (m)	Area Below Haunch Grade (m²)	Maximum Exterior Width (m)	Invert Thickness (mm)	Crown Thickness (mm)	Concrete Area Invert (m²)	Arch (m²)	Total (m²)	Concrete Weight (kN/m)	Backfill Properties Average Backfill Elevation (m)	Depth on Centerline (m)	Saturated Unit Weight Crown (kN/m³)	Saturated Unit Weight Arch Leg (kN/m³)	Backfill Weight (kN/m)	Backfill Shear Resistance (kN/m)	Total Resistance R (kN/m)	Aqueduct Cross-section Area Inside (m²)	Total (m²)	Buoyant Force B (kN/m)	Resisting Ratio R / B	Force to Resist (R / B<1) (kN/m)	Granular Ballast Minimum Depth Required (m)	Repair Length (m)	Design Depth (m)	Additional Resistance (kN/m)	New Total Resistance R (kN/m)	New Resisting Ratio R / B
85.711	BO	1151	318.85	316.44	316.29	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.84	0.998	12.38	12.38	[illegible]	0	55.18	4.64	6.58	67.76	0.81	12.6	[illegible]	13.75	0.90	[illegible]	76.75	1.13
85.720	BO	1150	318.85	316.45	316.30	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.76	0.902	11.22	11.22	[illegible]	0	47.65	4.64	6.58	67.76	0.70	20.1	[illegible]	13.71	0.90	[illegible]	69.22	1.02
85.728	BO	1149	318.84	316.44	316.29	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.84	0.995	11.22	11.22	[illegible]	0	48.03	4.64	6.58	67.76	0.71	19.7	[illegible]	13.78	0.90	[illegible]	69.60	1.03
85.737	BO	1148	318.86	316.46	316.30	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.91	1.053	11.22	11.22	[illegible]	0	48.26	4.64	6.58	67.76	0.71	19.5	[illegible]	13.74	0.90	[illegible]	69.83	1.03
85.745	BO	1147	318.86	316.45	316.30	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.58	0.721	11.22	11.22	[illegible]	0	46.88	4.64	6.58	67.76	0.69	20.9	[illegible]	13.66	0.90	[illegible]	68.45	1.01
85.754	BO	1146	318.86	316.46	316.30	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	320.06	1.200	11.22	11.22	[illegible]	0	48.85	4.64	6.58	67.76	0.72	18.9	[illegible]	13.68	0.90	[illegible]	70.42	1.04
85.763	BO	1145	318.86	316.46	316.30	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.89	1.031	11.22	11.22	[illegible]	0	48.15	4.64	6.58	67.76	0.71	19.6	[illegible]	13.77	0.90	[illegible]	69.72	1.03
85.771	BO	1144	318.87	316.46	316.31	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	320.06	1.193	11.22	11.22	[illegible]	0	48.81	4.64	6.58	67.76	0.72	18.9	[illegible]	13.62	0.90	[illegible]	70.38	1.04
85.780	BO	1143	318.86	316.46	316.30	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.85	0.992	11.22	11.22	[illegible]	0	47.97	4.64	6.58	67.76	0.71	19.8	[illegible]	13.78	0.90	[illegible]	69.54	1.03
85.788	BO	1142	318.86	316.46	316.30	2.251	2.657	0.923	3.658	152	152	0.62	1.33	1.94	39.33	319.63	0.772	11.21	11.21	[illegible]	0	47.06	4.64	6.58	67.76	0.69	20.7	[illegible]	13.72	0.90	[illegible]	68.63	1.01
85.797	BP	1141	318.87	316.47	316.31	2.251	2.556	0.483	4.064	152	152	0.80	1.33	2.13	43.09	319.77	0.896	11.21	11.21	[illegible]	0	51.79	4.64	6.76	69.67	0.74	17.9	[illegible]	13.77	0.90	[illegible]	75.75	1.09
85.805	BP	1140	318.88	316.48	316.32	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.04	1.164	11.21	11.21	[illegible]	0	53.96	4.64	6.76	69.67	0.77	15.7	[illegible]	13.68	0.75	[illegible]	73.93	1.06
85.814	BP	1139	318.88	316.48	316.32	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.64	0.763	11.21	11.21	[illegible]	0	52.13	4.64	6.76	69.67	0.75	17.5	[illegible]	13.77	0.75	[illegible]	72.10	1.03
85.822	BP	1138	318.89	316.48	316.33	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.88	0.989	11.18	11.18	[illegible]	0	52.93	4.64	6.76	69.67	0.76	16.7	[illegible]	13.70	0.75	[illegible]	72.90	1.05
85.831	BP	1137	318.89	316.48	316.33	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.97	1.088	11.15	11.15	[illegible]	0	53.14	4.64	6.76	69.67	0.76	16.5	[illegible]	13.70	0.75	[illegible]	73.12	1.05
85.839	BP	1136	318.88	316.48	316.33	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.39	0.512	11.12	11.12	[illegible]	0	50.46	4.64	6.76	69.67	0.72	19.2	[illegible]	13.72	0.75	[illegible]	70.43	1.01
85.848	BP	1135	318.89	316.49	316.33	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.99	1.101	11.09	11.09	[illegible]	0	52.74	4.64	6.76	69.67	0.76	16.9	[illegible]	13.72	0.75	[illegible]	72.71	1.04
85.856	BP	1134	318.89	316.48	316.33	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.73	0.846	11.06	11.06	[illegible]	0	51.47	4.64	6.76	69.67	0.74	18.2	[illegible]	13.73	0.75	[illegible]	71.44	1.03
85.865	BP	1133	318.89	316.49	316.33	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.93	1.038	11.03	11.03	[illegible]	0	52.02	4.64	6.76	69.67	0.75	17.6	[illegible]	13.76	0.75	[illegible]	71.99	1.03
85.871	B rd. x'ing	1132	318.99	316.50	316.29	2.251	2.692	0.922	4.343	203	238	0.91	2.06	2.97	60.18	319.90	0.912	11.01	11.01	[illegible]	0	68.76	4.64	7.61	78.37	0.88	9.6	[illegible]	9.19	0.75	[illegible]	90.10	1.15
85.879	BP	1131	318.90	316.49	316.34	2.251	2.556	0.483	4.064	152	152	0.80	1.33	2.13	43.09	320.02	1.119	10.98	10.98	[illegible]	0	51.16	4.64	6.76	69.67	0.73	18.5	[illegible]	13.64	0.75	[illegible]	71.13	1.02
85.888	BP	1130	318.90	316.50	316.34	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.11	1.209	10.94	10.94	[illegible]	0	52.05	4.64	6.76	69.67	0.75	17.6	[illegible]	13.73	0.75	[illegible]	72.02	1.03
85.896	BP	1129	318.91	316.51	316.36	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.12	1.209	10.91	10.91	[illegible]	0	51.81	4.64	6.76	69.67	0.74	17.9	[illegible]	13.70	0.75	[illegible]	71.78	1.03
85.905	BP	1128	318.93	316.53	316.38	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.85	0.911	10.88	10.88	[illegible]	0	50.53	4.64	6.76	69.67	0.73	19.1	[illegible]	13.70	0.75	[illegible]	70.50	1.01
85.913	BP	1127	318.94	316.54	316.38	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.04	1.096	10.89	10.89	[illegible]	0	51.24	4.64	6.76	69.67	0.74	18.4	[illegible]	13.75	0.75	[illegible]	71.21	1.02
85.922	BP	1126	318.95	316.54	316.39	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.55	0.602	10.90	10.90	[illegible]	0	49.56	4.64	6.76	69.67	0.71	20.1	[illegible]	13.69	0.75	[illegible]	69.53	1.00
85.930	BP	1125	318.95	316.54	316.39	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.11	1.159	10.91	10.91	[illegible]	0	51.61	4.64	6.76	69.67	0.74	18.1	[illegible]	13.71	0.75	[illegible]	71.58	1.03
85.939	BP	1124	318.95	316.55	316.40	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.96	1.001	10.92	10.92	[illegible]	0	51.11	4.64	6.76	69.67	0.73	18.6	[illegible]	13.76	0.75	[illegible]	71.08	1.02
85.947	BP	1123	318.95	316.55	316.40	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.84	0.885	10.93	10.93	[illegible]	0	50.75	4.64	6.76	69.67	0.73	18.9	[illegible]	13.67	0.75	[illegible]	70.72	1.02
85.956	BP	1122	318.96	316.56	316.41	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.96	0.995	10.94	10.94	[illegible]	0	51.22	4.64	6.76	69.67	0.74	18.5	[illegible]	13.73	0.75	[illegible]	71.19	1.02
85.964	BP	1121	318.97	316.56	316.41	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.12	1.149	10.95	10.95	[illegible]	0	51.86	4.64	6.76	69.67	0.74	17.8	[illegible]	13.76	0.75	[illegible]	71.83	1.03
85.973	BP	1120	318.97	316.57	316.41	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.91	0.935	10.96	10.96	[illegible]	0	51.13	4.64	6.76	69.67	0.73	18.5	[illegible]	13.68	0.75	[illegible]	71.10	1.02
85.981	BP	1119	318.97	316.57	316.42	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.94	0.966	10.97	10.97	[illegible]	0	51.31	4.64	6.76	69.67	0.74	18.4	[illegible]	13.70	0.75	[illegible]	71.28	1.02
85.990	BP	1118	318.98	316.58	316.42	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.13	1.145	10.98	10.98	[illegible]	0	52.06	4.64	6.76	69.67	0.75	17.6	[illegible]	13.74	0.75	[illegible]	72.03	1.03
85.998	BP	1117	318.99	316.59	316.44	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.10	1.111	10.97	10.97	[illegible]	0	51.90	4.64	6.76	69.67	0.74	17.8	[illegible]	13.71	0.75	[illegible]	71.87	1.03
86.007	BP	1116	318.99	316.58	316.43	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.20	1.211	10.97	10.97	[illegible]	0	52.24	4.64	6.76	69.67	0.75	17.4	[illegible]	13.74	0.75	[illegible]	72.21	1.04
86.015	BP	1115	318.99	316.59	316.43	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.03	1.039	10.96	10.96	[illegible]	0	51.57	4.64	6.76	69.67	0.74	18.1	[illegible]	13.69	0.75	[illegible]	71.54	1.03
86.024	BP	1114	318.99	316.59	316.44	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.09	1.092	10.96	10.96	[illegible]	0	51.73	4.64	6.76	69.67	0.74	17.9	[illegible]	13.70	0.75	[illegible]	71.70	1.03
86.033	BP	1113	319.00	316.60	316.44	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.86	0.863	14.98	14.98	[illegible]	0	78.11	4.64	6.76	69.67	1.12	0.0	[illegible]	0.00	0.75	[illegible]	98.08	1.41
86.041	BP	1112	319.00	316.60	316.45	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.98	0.979	13.03	13.03	[illegible]	0	66.12	4.64	6.76	69.67	0.95	3.6	[illegible]	13.71	0.75	[illegible]	86.09	1.24
86.050	BP	1111	319.01	316.61	316.45	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.06	1.048	13.15	13.15	[illegible]	0	67.70	4.64	6.76	69.67	0.97	2.0	[illegible]	13.70	0.75	[illegible]	87.67	1.26
86.058	BP	1110	319.01	316.61	316.45	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.15	1.140	13.27	13.27	[illegible]	0	69.62	4.64	6.76	69.67	1.00	0.1	[illegible]	13.76	0.75	[illegible]	89.59	1.29
86.067	BP	1109	319.01	316.61	316.46	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.20	1.190	13.38	13.38	[illegible]	0	71.09	4.64	6.76	69.67	1.02	0.0	[illegible]	0.00	0.75	[illegible]	91.06	1.31
86.075	BP	1108	319.02	316.61	316.46	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.92	0.899	13.50	13.50	[illegible]	0	68.53	4.64	6.76	69.67	0.98	1.1	[illegible]	13.70	0.75	[illegible]	88.50	1.27
86.084	BP	1107	319.02	316.62	316.46	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.13	1.105	13.62	13.62	[illegible]	0	71.90	4.64	6.76	69.67	1.03	0.0	[illegible]	0.00	0.75	[illegible]	91.87	1.32
86.092	BP	1106	319.03	316.62	316.47	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.13	1.104	14.74	14.74	[illegible]	0	80.38	4.64	6.76	69.67	1.15	0.0	[illegible]	0.00	0.75	[illegible]	100.35	1.44
86.101	BP	1105	319.03	316.63	316.47	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.09	1.061	15.86	15.86	[illegible]	0	88.00	4.64	6.76	69.67	1.26	0.0	[illegible]	0.00	0.75	[illegible]	107.97	1.55
86.109	BP	1104	319.03	316.63	316.48	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.09	1.056	16.99	16.99	[illegible]	0	96.21	4.64	6.76	69.67	1.38	0.0	[illegible]	0.00	0.75	[illegible]	116.18	1.67
86.118	BP	1103	319.04	316.64	316.48	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.15	1.106	18.11	18.11	[illegible]	0	105.87	4.64	6.76	69.67	1.52	0.0	[illegible]	0.00	0.75	[illegible]	125.85	1.81
86.126	BP	1102	319.03	316.63	316.48	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.95	0.914	19.23	19.23	[illegible]	0	108.48	4.64	6.76	69.67	1.56	0.0	[illegible]	0.00	0.75	[illegible]	128.45	1.84
86.135	BP	1101	319.04	316.64	316.49	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.15	1.101	20.35	20.35	[illegible]	0	122.66	4.64	6.76	69.67	1.76	0.0	[illegible]	0.00	0.75	[illegible]	142.63	2.05
86.143	BP	1100	319.05	316.64	316.49	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.06	1.010	21.48	21.48	[illegible]	0	127.71	4.64	6.76	69.67	1.83	0.0	[illegible]	0.00	0.75	[illegible]	147.68	2.12
86.152	BP	1099	319.07	316.67	316.51	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.11	1.043	18.22	18.22	[illegible]	0	104.94	4.64	6.76	69.67	1.51	0.0	[illegible]	0.00	0.75	[illegible]	124.91	1.79
86.160	BP	1098	319.07	316.67	316.52	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.24	1.165	14.95	14.95	[illegible]	0	82.95	4.64	6.76	69.67	1.19	0.0	[illegible]	0.00	0.75	[illegible]	102.92	1.48
86.169	BP	1097	319.08	316.68	316.52	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.08	1.003	13.48	13.48	[illegible]	0	69.61	4.64	6.76	69.67	1.00	0.1	[illegible]	13.72	0.75	[illegible]	89.58	1.29
86.177	BP	1096	319.08	316.68	316.52	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.14	1.058	12.01	12.01	[illegible]	0	59.38	4.64	6.76	69.67	0.85	10.3	[illegible]	13.75	0.75	[illegible]	79.35	1.14
86.186	BP	1095	319.11	316.71	316.55	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.12	1.006	11.91	11.91	[illegible]	0	58.28	4.64	6.76	69.67	0.84	11.4	[illegible]	13.67	0.75	[illegible]	78.25	1.12
86.194	BP	1094	319.11	316.71	316.56	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.03	0.921	11.81	11.81	[illegible]	0	56.99	4.64	6.76	69.67	0.82	12.7	[illegible]	13.74	0.75	[illegible]	76.96	1.10
86.203	BP	1093	319.12	316.72	316.56	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.73	0.613	11.70	11.70	[illegible]	0	54.38	4.64	6.76	69.67	0.78	15.3	[illegible]	13.77	0.75	[illegible]	74.35	1.07
86.212	BP	1092	319.12	316.71	316.56	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.97	0.851	11.60	11.60	[illegible]	0	55.16	4.64	6.76	69.67	0.79	14.5	[illegible]	13.68	0.75	[illegible]	75.13	1.08
86.220	BP	1091	319.13	316.72	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.99	0.867	11.50	11.50	[illegible]	0	54.55	4.64	6.76	69.67	0.78	15.1	[illegible]	13.76	0.75	[illegible]	74.52	1.07
86.229	BP	1090	319.13	316.73	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.16	1.029	11.40	11.40	[illegible]	0	54.69	4.64	6.76	69.67	0.78	15.0	[illegible]	13.72	0.75	[illegible]	74.66	1.07
86.237	BP	1089	319.13	316.73	316.58	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.01	0.876	11.29	11.29	[illegible]	0	53.21	4.64	6.76	69.67	0.76	16.5	[illegible]	13.70	0.75	[illegible]	73.18	1.05
86.246	BP	1088	319.13	316.73	316.58	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.90	0.767	11.19	11.19	[illegible]	0	52.02	4.64	6.76	69.67	0.75	17.6	[illegible]	13.72	0.75	[illegible]	72.00	1.03
86.254	BP	1087	319.13	316.72	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.94	0.814	11.09	11.09	[illegible]	0	51.56	4.64	6.76	69.67	0.74	18.1	[illegible]	13.72	0.75	[illegible]	71.53	1.03
86.263	BP	1086	319.12	316.72	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.93	0.811	10.99	10.99	[illegible]	0	50.87	4.64	6.76	69.67	0.73	18.8	[illegible]	13.74	0.75	[illegible]	70.84	1.02
86.271	BP	1085	319.12	316.72	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.08	0.957	10.89	10.89	[illegible]	0	50.71	4.64	6.76	69.67	0.73	19.0	[illegible]	13.70	0.75	[illegible]	70.68	1.01
86.280	BP	1084	319.11	316.71	316.55	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.12	1.008	10.92	10.92	[illegible]	0	51.13	4.64	6.76	69.67	0.73	18.5	[illegible]	13.72	0.75	[illegible]	71.10	1.02
86.288	BP	1083	319.11	316.71	316.56	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.10	0.986	10.95	10.95	[illegible]	0	51.29	4.64	6.76	69.67	0.74	18.4	[illegible]	13.67	0.75	[illegible]	71.26	1.02
86.297	BP	1082	319.11	316.71	316.56	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.28	1.161	10.99	10.99	[illegible]	0	52.21	4.64	6.76	69.67	0.75	17.5	[illegible]	13.78	0.75	[illegible]	72.18	1.04
86.305	BP	1081	319.12	316.72	316.56	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.31	1.191	11.02	11.02	[illegible]	0	52.59	4.64	6.76	69.67	0.75	17.1	[illegible]	13.68	0.75	[illegible]	72.56	1.04
86.314	BP	1080	319.12	316.72	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.32	1.193	11.06	11.06	[illegible]	0	52.87	4.64	6.76	69.67	0.76	16.8	[illegible]	13.72	0.75	[illegible]	72.84	1.05
86.322	BP	1079	319.13	316.73	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.24	1.114	11.09	11.09	[illegible]	0	52.80	4.64	6.76	69.67	0.76	16.9	[illegible]	13.77	0.75	[illegible]	72.77	1.04
86.331	BP	1078	319.13	316.73	316.57	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.28	1.146	11.12	11.12	[illegible]	0	53.20	4.64	6.76	69.67	0.76	16.5	[illegible]	13.69	0.75	[illegible]	73.17	1.05
86.339	BP	1077	319.14	316.74	316.58	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.25	1.109	11.16	11.16	[illegible]	0	53.30	4.64	6.76	69.67	0.77	16.4	[illegible]	13.72	0.75	[illegible]	73.27	1.05
86.348	BP	1076	319.14	316.73	316.58	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.55	0.409	11.19	11.19	[illegible]	0	50.41	4.64	6.76	69.67	0.72	19.3	[illegible]	13.74	0.75	[illegible]	70.38	1.01
86.356	BP	1075	319.15	316.74	316.59	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.01	0.864	11.23	11.23	[illegible]	0	52.68	4.64	6.76	69.67	0.76	17.0	[illegible]	13.68	0.75	[illegible]	72.65	1.04
86.365	BP	1074	319.14	316.74	316.59	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.15	1.005	11.26	11.26	[illegible]	0	53.58	4.64	6.76	69.67	0.77	16.1	[illegible]	13.73	0.75	[illegible]	73.55	1.06
86.373	BP	1073	319.16	316.76	316.61	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.05	0.892	11.22	11.22	[illegible]	0	52.79	4.64	6.76	69.67	0.76	16.9	[illegible]	13.64	0.75	[illegible]	72.76	1.04
86.382	BP	1072	319.16	316.76	316.60	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.43	1.267	11.19	11.19	[illegible]	0	54.22	4.64	6.76	69.67	0.78	15.5	[illegible]	13.78	0.75	[illegible]	74.19	1.06
86.391	BP	1071	319.16	316.76	316.61	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.35	1.190	11.15	11.15	[illegible]	0	53.59	4.64	6.76	69.67	0.77	16.1	[illegible]	13.71	0.75	[illegible]	73.56	1.06
86.399	BP	1070	319.18	316.77	316.62	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.32	1.144	11.11	11.11	[illegible]	0	53.10	4.64	6.76	69.67	0.76	16.6	[illegible]	13.73	0.75	[illegible]	73.07	1.05
86.408	BP	1069	319.18	316.77	316.62	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.97	0.796	11.08	11.08	[illegible]	0	51.39	4.64	6.76	69.67	0.74	18.3	[illegible]	13.75	0.75	[illegible]	71.36	1.02
86.416	BP	1068	319.18	316.78	316.62	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.28	1.099	11.04	11.04	[illegible]	0	52.36	4.64	6.76	69.67	0.75	17.3	[illegible]	13.75	0.75	[illegible]	72.33	1.04
86.425	BP	1067	319.19	316.79	316.63	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.18	0.991	11.00	11.00	[illegible]	0	51.67	4.64	6.76	69.67	0.74	18.0	[illegible]	13.69	0.75	[illegible]	71.64	1.03

SHOAL LAKE AQUEDUCT BUOYANCY ASSESSMENT - Boggy River Stretch (Mile 85.0 to 88.5) - Partial Safety Factor Method Using OHBDC (1991) Load and Resistance Factors

General Buoyancy Equation: [$\phi_{backfill}$ (backfill weight) + ϕ_{shear} (backfill shear resistance) + $\phi_{concrete}$ (weight of aqueduct) + $\phi_{ballast}$ (granular ballast weight)] ≥ α (buoyant force)

Backfill Notes
- (light shading) measured or predicted unit weight
- (dark shading) submerged unit weights
- (no shading) bulk unit weights (above crown only)

Backfill Shear Resistance: Depth of Compacted Backfill Above Invert
- 0.0 = peat backfill (no shear resistance)
- 1.0 = clay layer (m) (depth contributing to shear resistance)

Concrete Unit Weight and Resistance Factor
- $\gamma_{Concrete}$ = 22.50 (kN/m³)
- Resistance Factor, ϕconcrete = 0.90

Backfill Resistance Factors
- Submerged Backfill, ϕsub. = 0.80
- Non-Submerged Crown Backfill, ϕbulk = 0.80
- Backfill Shear, ϕshear = 0.50
- Undrained Shear Strength = 12.0 (kPa)

Buoyancy Load Parameters
- γ_{water} = 9.81 (kN/m³)
- Buoyancy Load Factor, α = 1.05

Granular Ballast Parameters
- $\gamma_{Ballast\ bulk}$ = 14.40 (kN/m³)
- Resistance Factor, ϕbulk = 0.80 Non-Submerged ballast
- $\gamma_{Ballast\ Saturated}$ = 18.00 (kN/m³)
- Resistance Factor, ϕsub = 0.80 Submerged Ballast

Aqueduct Mileage	Aqueduct Segment Section Type	Aqueduct Segment No.	Aqueduct Survey Crown Elevation (m)	Invert Elevations Inside Aqueduct (m)	Invert Elevations Foundation Side (m)	Aqueduct Dimensions Inside Height (m)	Outside Height (m)	Area Below Haunch Grade (m²)	Maximum Exterior Width (m)	Invert Thickness (mm)	Crown Thickness (mm)	Concrete Area Invert (m²)	Arch (m²)	Total (m²)	Concrete Weight (kN/m)	Backfill Properties Average Backfill Elevation (m)	Depth on Centerline (m)	Saturated Unit Weight Crown (kN/m³)	Saturated Unit Weight Arch Leg (kN/m³)	Backfill Weight (kN/m)	Backfill Shear Resistance (kN/m)	Total Resistance R (kN/m)	Aqueduct Cross-section Area Inside (m²)	Total (m²)	Buoyant Force B (kN/m)	Resisting Ratio R/B	Force to Resist (R/B<1) (kN/m)	Granular Ballast Minimum Depth Required (m)	Repair Length (m)	Design Depth (m)	Additional Resistance (kN/m)	New Total Resistance R (kN/m)	New Resisting Ratio R/B
86.433	BP	1066	319.19	316.79	316.64	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.34	1.149	10.97	10.97	[illegible]	0	51.99	4.64	6.76	69.67	0.75	17.7	[illegible]	13.80	0.75	[illegible]	71.96	1.03
86.442	BP	1065	319.20	316.79	316.64	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.38	1.178	10.93	10.93	[illegible]	0	51.82	4.64	6.76	69.67	0.74	17.9	[illegible]	13.68	0.75	[illegible]	71.79	1.03
86.450	BP	1064	319.19	316.79	316.63	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.38	1.190	10.89	10.89	[illegible]	0	51.57	4.64	6.76	69.67	0.74	18.1	[illegible]	13.72	0.75	[illegible]	71.54	1.03
86.459	BP	1063	319.21	316.80	316.65	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.72	0.508	11.05	11.05	[illegible]	0	50.06	4.64	6.76	69.67	0.72	19.6	[illegible]	13.74	0.75	[illegible]	70.03	1.01
86.467	BP	1062	319.21	316.80	316.65	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.09	0.880	11.21	11.21	[illegible]	0	52.64	4.64	6.76	69.67	0.76	17.0	[illegible]	13.71	0.75	[illegible]	72.61	1.04
86.476	BP	1061	319.21	316.81	316.66	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.38	1.164	11.37	11.37	[illegible]	0	55.16	4.64	6.76	69.67	0.79	14.5	[illegible]	13.69	0.75	[illegible]	75.13	1.08
86.484	BP	1060	319.22	316.82	316.67	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.48	1.257	11.52	11.52	[illegible]	0	56.90	4.64	6.76	69.67	0.82	12.8	[illegible]	13.75	0.75	[illegible]	76.87	1.10
86.493	BP	1059	319.22	316.81	316.66	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.30	1.078	11.68	11.68	[illegible]	0	57.08	4.64	6.76	69.67	0.82	12.6	[illegible]	13.69	0.75	[illegible]	77.05	1.11
86.501	BP	1058	319.23	316.82	316.67	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.48	1.248	11.84	11.84	[illegible]	0	59.38	4.64	6.76	69.67	0.85	10.3	[illegible]	13.75	0.75	[illegible]	79.36	1.14
86.510	BP	1057	319.24	316.83	316.68	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.37	1.135	12.00	12.00	[illegible]	0	59.85	4.64	6.76	69.67	0.86	9.8	[illegible]	13.69	0.75	[illegible]	79.82	1.15
86.518	BP	1056	319.24	316.84	316.68	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.42	1.176	12.16	12.16	[illegible]	0	61.38	4.64	6.76	69.67	0.88	8.3	[illegible]	13.75	0.75	[illegible]	81.35	1.17
86.527	BP	1055	319.25	316.84	316.69	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.36	1.116	12.32	12.32	[illegible]	0	62.11	4.64	6.76	69.67	0.89	7.6	[illegible]	13.69	0.75	[illegible]	82.08	1.18
86.536	BP	1054	319.24	316.84	316.69	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.22	0.976	12.47	12.47	[illegible]	0	62.11	4.64	6.76	69.67	0.89	7.6	[illegible]	13.75	0.75	[illegible]	82.08	1.18
86.544	BP	1053	319.25	316.84	316.69	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.13	0.877	12.63	12.63	[illegible]	0	62.33	4.64	6.76	69.67	0.89	7.3	[illegible]	13.70	0.75	[illegible]	82.30	1.18
86.553	BP	1052	319.26	316.85	316.70	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.35	1.092	12.79	12.79	[illegible]	0	65.48	4.64	6.76	69.67	0.94	4.2	[illegible]	13.74	0.75	[illegible]	85.46	1.23
86.561	BP	1051	319.26	316.86	316.71	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.19	0.928	12.05	12.05	[illegible]	0	58.70	4.64	6.76	69.67	0.84	11.0	[illegible]	13.73	0.75	[illegible]	78.67	1.13
86.570	BP	1050	319.25	316.85	316.70	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.19	0.933	11.30	11.30	[illegible]	0	53.52	4.64	6.76	69.67	0.77	16.2	[illegible]	13.73	0.75	[illegible]	73.49	1.05
86.578	BP	1049	319.25	316.85	316.69	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.36	1.110	11.83	11.83	[illegible]	0	58.35	4.64	6.76	69.67	0.84	11.3	[illegible]	13.72	0.75	[illegible]	78.32	1.12
86.587	BP	1048	319.25	316.85	316.70	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	319.99	0.735	12.35	12.35	[illegible]	0	59.23	4.64	6.76	69.67	0.85	10.4	[illegible]	13.63	0.75	[illegible]	79.20	1.14
86.595	BP	1047	319.26	316.86	316.70	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.10	0.837	12.94	12.94	[illegible]	0	63.99	4.64	6.76	69.67	0.92	5.7	[illegible]	13.74	0.75	[illegible]	83.96	1.21
86.604	BP	1046	319.27	316.86	316.71	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.23	0.964	13.52	13.52	[illegible]	0	69.43	4.64	6.76	69.67	1.00	0.2	[illegible]	13.76	0.75	[illegible]	89.40	1.28
86.612	BP	1045	319.27	316.87	316.72	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.27	0.997	14.11	14.11	[illegible]	0	74.05	4.64	6.76	69.67	1.06	0.0	[illegible]	0.00	0.75	[illegible]	94.02	1.35
86.621	BP	1044	319.26	316.85	316.70	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.49	1.233	14.69	14.69	[illegible]	0	82.03	4.64	6.76	69.67	1.18	0.0	[illegible]	0.00	0.75	[illegible]	102.00	1.46
86.629	BP	1043	319.27	316.87	316.71	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.22	0.944	15.28	15.28	[illegible]	0	81.57	4.64	6.76	69.67	1.17	0.0	[illegible]	0.00	0.75	[illegible]	101.54	1.46
86.638	BP	1042	319.28	316.88	316.72	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.08	0.805	15.87	15.87	[illegible]	0	82.95	4.64	6.76	69.67	1.19	0.0	[illegible]	0.00	0.75	[illegible]	102.92	1.48
86.646	BP	1041	319.28	316.88	316.73	2.251	2.657	0.923	4.064	152	152	0.80	1.33	2.13	43.09	320.49	1.204	16.46	16.46	[illegible]	0	95.45	4.64	6.76	69.67	1.37	0.0	[illegible]	0.00	0.75	[illegible]	115.42	1.66
86.655	BPM	1040	319.28	316.87	316.72	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.51	1.237	14.45	14.45	[illegible]	0	79.59	4.64	6.92	71.29	1.12	0.0	[illegible]	0.00	0.75	[illegible]	99.56	1.40
86.663	BPM	1039	319.29	316.89	316.73	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.50	1.213	12.44	12.44	[illegible]	0	64.96	4.64	6.92	71.29	0.91	6.3	[illegible]	13.76	0.75	[illegible]	84.93	1.19
86.672	BPM	1038	319.29	316.88	316.73	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.21	0.923	11.24	11.24	[illegible]	0	55.08	4.64	6.92	71.29	0.77	16.2	[illegible]	13.69	0.75	[illegible]	75.05	1.05
86.680	BPM	1037	319.30	316.89	316.74	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.52	1.224	11.12	11.12	[illegible]	0	55.62	4.64	6.92	71.29	0.78	15.7	[illegible]	13.78	0.75	[illegible]	75.59	1.06
86.689	BPM	1036	319.31	316.90	316.75	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.57	1.266	11.00	11.00	[illegible]	0	54.93	4.64	6.92	71.29	0.77	16.4	[illegible]	13.76	0.75	[illegible]	74.90	1.05
86.697	BPM	1035	319.29	316.89	316.73	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.57	1.281	11.03	11.03	[illegible]	0	55.23	4.64	6.92	71.29	0.77	16.1	[illegible]	13.68	0.75	[illegible]	75.21	1.05
86.706	BPM	1034	319.29	316.89	316.73	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.53	1.241	11.07	11.07	[illegible]	0	55.31	4.64	6.92	71.29	0.78	16.0	[illegible]	13.73	0.75	[illegible]	75.28	1.06
86.714	BPM	1033	319.30	316.90	316.75	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.54	1.236	11.10	11.10	[illegible]	0	55.53	4.64	6.92	71.29	0.78	15.8	[illegible]	13.67	0.75	[illegible]	75.50	1.06
86.723	BPM	1032	319.31	316.91	316.76	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.25	0.938	11.13	11.13	[illegible]	0	54.50	4.64	6.92	71.29	0.76	16.8	[illegible]	13.78	0.75	[illegible]	74.47	1.04
86.731	BPM	1031	319.31	316.91	316.76	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.34	1.022	11.17	11.17	[illegible]	0	55.08	4.64	6.92	71.29	0.77	16.2	[illegible]	13.68	0.75	[illegible]	75.05	1.05
86.740	BPM	1030	319.31	316.91	316.75	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.58	1.274	11.24	11.24	[illegible]	0	56.73	4.64	6.92	71.29	0.80	14.6	[illegible]	13.81	0.75	[illegible]	76.70	1.08
86.748	BPM	1029	319.32	316.92	316.76	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.37	1.048	11.32	11.32	[illegible]	0	56.16	4.64	6.92	71.29	0.79	15.1	[illegible]	13.61	0.75	[illegible]	76.13	1.07
86.757	BPM	1028	319.32	316.92	316.76	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.72	1.395	11.39	11.39	[illegible]	0	58.42	4.64	6.92	71.29	0.82	12.9	[illegible]	13.81	0.75	[illegible]	78.40	1.10
86.765	BPM	1027	319.33	316.93	316.77	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.80	1.474	11.46	11.46	[illegible]	0	59.42	4.64	6.92	71.29	0.83	11.9	[illegible]	13.79	0.75	[illegible]	79.39	1.11
86.774	BPM	1026	319.32	316.92	316.77	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.77	1.440	11.54	11.54	[illegible]	0	59.81	4.64	6.92	71.29	0.84	11.5	[illegible]	13.60	0.75	[illegible]	79.78	1.12
86.782	BPM	1025	319.33	316.93	316.78	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.68	1.351	11.61	11.61	[illegible]	0	59.86	4.64	6.92	71.29	0.84	11.4	[illegible]	13.72	0.75	[illegible]	79.83	1.12
86.791	BPM	1024	319.32	316.92	316.76	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.70	1.383	11.68	11.68	[illegible]	0	60.62	4.64	6.92	71.29	0.85	10.7	[illegible]	13.77	0.75	[illegible]	80.59	1.13
86.799	BPM	1023	319.34	316.94	316.79	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.70	1.358	11.76	11.76	[illegible]	0	61.02	4.64	6.92	71.29	0.86	10.3	[illegible]	13.66	0.75	[illegible]	80.99	1.14
86.808	BPM	1022	319.34	316.94	316.79	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.42	1.080	11.83	11.83	[illegible]	0	59.74	4.64	6.92	71.29	0.84	11.5	[illegible]	13.74	0.75	[illegible]	79.71	1.12
86.816	BPM	1021	319.35	316.94	316.79	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.85	1.498	11.90	11.90	[illegible]	0	63.09	4.64	6.92	71.29	0.88	8.2	[illegible]	13.75	0.75	[illegible]	83.06	1.17
86.825	BPM	1020	319.35	316.95	316.80	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.75	1.397	11.98	11.98	[illegible]	0	62.97	4.64	6.92	71.29	0.88	8.3	[illegible]	13.78	0.75	[illegible]	82.94	1.16
86.833	BPM	1019	319.35	316.95	316.80	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.80	1.443	12.05	12.05	[illegible]	0	63.86	4.64	6.92	71.29	0.90	7.4	[illegible]	13.68	0.75	[illegible]	83.83	1.18
86.842	BPM	1018	319.36	316.96	316.80	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.87	1.516	12.21	12.21	[illegible]	0	65.64	4.64	6.92	71.29	0.92	5.6	[illegible]	13.69	0.75	[illegible]	85.62	1.20
86.847	BPM	1017	319.36	316.96	316.80	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.63	1.274	12.31	12.31	[illegible]	0	64.52	4.64	6.92	71.29	0.91	6.8	[illegible]	9.23	0.75	[illegible]	84.49	1.19
86.856	BPM	1016	319.37	316.97	316.82	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.46	1.082	12.46	12.46	[illegible]	0	64.00	4.64	6.92	71.29	0.90	7.3	[illegible]	13.68	0.75	[illegible]	83.97	1.18
86.864	BPM	1015	319.37	316.96	316.81	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.73	1.358	12.62	12.62	[illegible]	0	67.55	4.64	6.92	71.29	0.95	3.7	[illegible]	13.74	0.75	[illegible]	87.52	1.23
86.873	BPM	1014	319.38	316.98	316.83	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.60	1.212	12.77	12.77	[illegible]	0	67.32	4.64	6.92	71.29	0.94	4.0	[illegible]	13.73	0.75	[illegible]	87.29	1.22
86.881	BPM	1013	319.39	316.99	316.84	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.63	1.236	12.93	12.93	[illegible]	0	68.66	4.64	6.92	71.29	0.96	2.6	[illegible]	13.73	0.75	[illegible]	88.63	1.24
86.890	BPM	1012	319.39	316.99	316.83	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.59	1.201	13.70	13.70	[illegible]	0	73.77	4.64	6.92	71.29	1.03	0.0	[illegible]	0.00	0.75	[illegible]	93.74	1.31
86.895	B rd xing	1011	319.49	317.00	316.80	2.251	2.692	0.922	4.343	203	238	0.91	2.06	2.97	60.18	320.75	1.261	14.22	14.22	[illegible]	0	97.14	4.64	7.61	78.37	1.24	0.0	[illegible]	0.00	0.75	21.34	118.48	1.51
86.904	BPM	1010	319.41	317.00	316.85	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.52	1.118	14.99	14.99	[illegible]	0	81.46	4.64	6.92	71.29	1.14	0.0	[illegible]	0.00	0.75	[illegible]	101.43	1.42
86.912	BPM	1009	319.41	317.00	316.85	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.98	1.575	16.70	16.70	[illegible]	0	103.29	4.64	6.92	71.29	1.45	0.0	[illegible]	0.00	0.75	[illegible]	123.27	1.73
86.921	BPM	1008	319.39	316.99	316.83	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.90	1.513	18.41	18.41	[illegible]	0	115.73	4.64	6.92	71.29	1.62	0.0	[illegible]	0.00	0.75	[illegible]	135.70	1.90
86.929	BPM	1007	319.41	317.01	316.86	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.91	1.495	17.81	17.81	[illegible]	0	110.45	4.64	6.92	71.29	1.55	0.0	[illegible]	0.00	0.75	[illegible]	130.42	1.83
86.938	BPM	1006	319.42	317.02	316.86	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.58	1.161	17.22	17.22	[illegible]	0	97.68	4.64	6.92	71.29	1.37	0.0	[illegible]	0.00	0.75	[illegible]	117.65	1.65
86.946	BPM	1005	319.42	317.02	316.87	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	321.00	1.576	16.63	16.63	[illegible]	0	102.76	4.64	6.92	71.29	1.44	0.0	[illegible]	0.00	0.75	[illegible]	122.73	1.72
86.949	BPM	1004	319.42	317.02	316.86	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.96	1.540	16.43	16.43	[illegible]	0	100.32	4.64	6.92	71.29	1.41	0.0	[illegible]	0.00	0.75	[illegible]	120.29	1.69
86.958	BPM	1003	319.42	317.02	316.87	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.99	1.568	16.49	16.49	[illegible]	0	101.43	4.64	6.92	71.29	1.42	0.0	[illegible]	0.00	0.75	[illegible]	121.40	1.70
86.966	BPM	1002	319.42	317.02	316.87	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.61	1.181	16.55	16.55	[illegible]	0	93.45	4.64	6.92	71.29	1.31	0.0	[illegible]	0.00	0.75	[illegible]	113.42	1.59
86.974	BPM	1001	319.43	317.03	316.88	2.251	2.556	0.483	4.064	152	152	0.83	1.46	2.29	46.27	320.88	1.452	16.61	16.61	[illegible]	0	99.85	4.64	6.92	71.29	1.40	0.0	[illegible]	0.00	0.75	[illegible]	119.82	1.68
86.980	RB	1	319.44	317.04	316.88	2.251	2.574	0.496	4.064	152	152	0.80	1.33	2.13	43.09	321.03	1.589	16.00	16.00	[illegible]	0	95.78	4.64	6.76	69.67	1.37	0.0	[illegible]	0.00	0.75	[illegible]	115.75	1.66
86.989	RPM	249	319.48	317.05	316.89	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.86	1.374	15.07	15.07	[illegible]	0	87.75	4.78	7.11	73.19	1.20	0.0	[illegible]	0.00	0.75	[illegible]	107.88	1.47
86.997	RPM	248	319.48	317.05	316.89	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.88	1.397	14.14	14.14	[illegible]	0	80.93	4.78	7.11	73.19	1.11	0.0	[illegible]	0.00	0.75	[illegible]	101.06	1.38
87.006	RPM	247	319.48	317.05	316.89	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.57	1.086	13.22	13.22	[illegible]	0	70.25	4.78	7.11	73.19	0.96	2.9	[illegible]	13.69	0.75	[illegible]	90.38	1.23
87.014	RPM	246	319.49	317.05	316.90	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.41	0.923	12.29	12.29	[illegible]	0	62.64	4.78	7.11	73.19	0.86	10.5	[illegible]	13.74	0.75	[illegible]	82.77	1.13
87.023	RPM	245	319.49	317.05	316.90	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.66	1.172	12.14	12.14	[illegible]	0	63.63	4.78	7.11	73.19	0.87	9.6	[illegible]	13.74	0.75	[illegible]	83.76	1.14
87.031	RPM	244	319.51	317.07	316.92	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.99	1.483	11.99	11.99	[illegible]	0	64.83	4.78	7.11	73.19	0.89	8.4	[illegible]	13.69	0.75	[illegible]	84.95	1.16
87.040	RPM	243	319.51	317.07	316.92	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.91	1.403	11.85	11.85	[illegible]	0	63.11	4.78	7.11	73.19	0.86	10.1	[illegible]	13.72	0.75	[illegible]	83.24	1.14
87.048	RPM	242	319.45	317.01	316.86	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	321.04	1.589	11.70	11.70	[illegible]	0	63.13	4.78	7.11	73.19	0.86	10.1	[illegible]	13.70	0.75	[illegible]	83.25	1.14
87.057	RPM	241	319.46	317.02	316.87	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.82	1.364	11.44	11.44	[illegible]	0	59.70	4.78	7.11	73.19	0.82	13.5	[illegible]	13.71	0.75	[illegible]	79.83	1.09
87.065	RPM	240	319.46	317.02	316.87	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.57	1.108	11.17	11.17	[illegible]	0	56.52	4.78	7.11	73.19	0.77	16.7	[illegible]	13.75	0.75	[illegible]	76.65	1.05
87.074	RPM	239	319.46	317.03	316.87	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.53	1.065	10.91	10.91	[illegible]	0	54.56	4.78	7.11	73.19	0.75	18.6	[illegible]	13.71	0.75	[illegible]	74.69	1.02
87.082	RPM	238	319.47	317.03	316.88	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.57	1.099	11.34	11.34	[illegible]	0	57.64	4.78	7.11	73.19	0.79	15.5	[illegible]	13.69	0.75	[illegible]	77.77	1.06
87.091	RPM	237	319.46	317.02	316.87	2.276	2.591	0.512	4.096	152	162	0.82	1.51	2.33	47.18	320.92	1.465	11.78	11.78	[illegible]	0	62.98	4.78	7.11	73.19	0.86	10.2	[illegible]	13.68	0.75	[illegible]	83.11	1.14
87.099	RRM	236	319.48	317.05	316.75	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.88	1.390	12.22	12.22	[illegible]	0	74.66	4.78	7.48	77.09	0.97	2.4	[illegible]	13.75	0.75	[illegible]	94.79	1.23
87.108	RRM	235	319.48	317.04	316.75	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.96	1.485	12.65	12.65	[illegible]	0	79.11	4.78	7.48	77.09	1.03	0.0	[illegible]	0.00	0.75	[illegible]	99.24	1.29
87.116	RRM	234	319.50	317.06	316.77	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.41	0.906	12.59	12.59	[illegible]	0	73.34	4.78	7.48	77.09	0.95	3.8	[illegible]	13.71	0.75	[illegible]	93.46	1.21
87.125	RRM	233	319.49	317.06	316.76	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.94	1.442	12.53	12.53	[illegible]	0	77.73	4.78	7.48	77.09	1.01	0.0	[illegible]	0.00	0.75	[illegible]	97.86	1.27
87.133	RRM	232	319.49	317.05	316.76	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.56	1.064	12.48	12.48	[illegible]	0	73.94	4.78	7.48	77.09	0.96	3.1	[illegible]	13.67	0.75	[illegible]	94.07	1.22

SHOAL LAKE AQUEDUCT BUOYANCY ASSESSMENT - Boggy River Stretch (Mile 85.0 to 88.5) - Partial Safety Factor Method Using OHBDC (1991) Load and Resistance Factors

General Buoyancy Equation: [$\phi_{backfill}$ (backfill weight) + ϕ_{shear} (backfill shear resistance) + $\phi_{concrete}$ (weight of aqueduct) + $\phi_{ballast}$ (granular ballast weight)] $\geq \alpha$ (buoyant force)

Backfill Notes
- measured or predicted unit weight
- submerged unit weights
- bulk unit weights (above crown only)

Backfill Shear Resistance:
Depth of Compacted Backfill Above Invert
- 0.0 = peat backfill (no shear resistance)
- 1.0 = clay layer (m) (depth contributing to shear resistance)

Concrete Unit Weight and Resistance Factor
- $\gamma_{Concrete}$ = 22.50 (kN/m³)
- Resistance Factor, ϕconcrete = 0.90

Backfill Resistance Factors
- Submerged Backfill, ϕsub. = 0.80
- Non-Submerged Crown Backfill, ϕbulk = 0.80
- Backfill Shear, ϕshear = 0.50
- Undrained Shear Strength = 12.0 (kPa)

Buoyancy Load Parameters
- γ_{water} = 9.81 (kN/m³)
- Buoyancy Load Factor, α = 1.05

Granular Ballast Parameters
- $\gamma_{Ballast\ bulk}$ = 14.40 (kN/m³)
- Resistance Factor, ϕbulk = 0.80 Non-Submerged ballast
- $\gamma_{Ballast\ Saturated}$ = 18.00 (kN/m³)
- Resistance Factor, ϕsub = 0.80 Submerged Ballast

Aqueduct Mileage	Aqueduct Segment Section Type	Aqueduct Segment No.	Aqueduct Survey Crown Elevation (m)	Invert Elevations Inside Aqueduct (m)	Invert Elevations Foundation Side (m)	Aqueduct Dimensions Inside Height (m)	Outside Height (m)	Area Below Haunch Grade (m²)	Maximum Exterior Width (m)	Invert Thickness (mm)	Crown Thickness (mm)	Concrete Area Invert (m²)	Arch (m²)	Total (m²)	Concrete Weight (kN/m)	Backfill Properties Average Backfill Elevation (m)	Depth on Centerline (m)	Saturated Unit Weight Crown (kN/m³)	Arch Leg (kN/m³)	Backfill Weight (kN/m)	Backfill Shear Resistance (kN/m)	Total Resistance R (kN/m)	Aqueduct Cross-section Area Inside (m²)	Total (m²)	Buoyant Force B (kN/m)	Resisting Ratio R/B	Force to Resist (R/B<1) (kN/m)	Granular Ballast Minimum Depth Required (m)	Repair Length (m)	Design Depth (m)	Additional Resistance (kN/m)	New Total Resistance R (kN/m)	New Resisting Ratio R/B
87.142	RRM	231	319.49	317.05	316.76	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.92	1.432	12.42	12.42	21.81	0	76.67	4.78	7.48	77.09	0.99	0.4	0.02	13.78	0.75	20.13	96.79	1.26
87.150	RRM	230	319.48	317.04	316.75	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.08	1.601	12.36	12.36	22.74	0	77.59	4.78	7.48	77.09	1.01	0.0	0.00	0.00	0.75	20.12	97.72	1.27
87.159	RRM	229	319.48	317.04	316.75	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.04	1.565	12.30	12.30	21.92	0	76.78	4.78	7.48	77.09	1.00	0.3	0.01	13.75	0.75	20.13	96.91	1.26
87.167	RRM	228	319.49	317.05	316.76	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.09	1.606	12.24	12.24	21.74	0	76.59	4.78	7.48	77.09	0.99	0.5	0.02	13.69	0.75	20.13	96.72	1.25
87.175	RRM	227	319.48	317.05	316.75	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.04	1.552	12.18	12.18	20.80	0	75.65	4.78	7.48	77.09	0.98	1.4	0.05	13.69	0.75	20.13	95.78	1.24
87.184	RRM	226	319.50	317.07	316.77	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.94	1.436	12.13	12.13	19.40	0	74.26	4.78	7.48	77.09	0.96	2.8	0.11	13.79	0.75	20.13	94.39	1.22
87.193	RRM	225	319.51	317.08	316.78	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.89	1.375	12.07	12.07	18.48	0	73.32	4.78	7.48	77.09	0.95	3.8	0.14	13.73	0.75	20.12	93.45	1.21
87.201	RRM	224	319.51	317.07	316.78	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.79	1.280	12.01	12.01	17.39	0	72.16	4.78	7.48	77.09	0.94	4.9	0.18	13.69	0.75	20.13	92.29	1.20
87.210	RRM	223	319.51	317.07	316.78	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.74	1.230	11.86	11.86	15.70	0	70.65	4.78	7.48	77.09	0.92	6.4	0.24	13.77	0.75	20.13	90.78	1.18
87.218	RRM	222	319.52	317.08	316.79	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.51	0.993	11.71	11.71	13.17	0	68.03	4.78	7.48	77.09	0.88	9.1	0.34	13.68	0.75	20.13	88.16	1.14
87.226	RRM	221	319.52	317.08	316.79	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.69	1.174	11.56	11.56	13.18	0	68.04	4.78	7.48	77.09	0.88	9.1	0.34	13.73	0.75	20.12	88.16	1.14
87.235	RRM	220	319.53	317.09	316.80	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.71	1.187	11.41	11.41	12.13	0	66.99	4.78	7.48	77.09	0.87	10.1	0.38	13.70	0.75	20.12	87.11	1.13
87.243	RRM	219	319.53	317.09	316.80	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.28	0.757	11.26	11.26	8.95	0	63.81	4.78	7.48	77.09	0.83	13.3	0.49	13.77	0.75	20.12	83.93	1.09
87.252	RRM	218	319.53	317.10	316.80	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.65	1.120	11.12	11.12	9.59	0	64.44	4.78	7.48	77.09	0.84	12.6	0.47	13.72	0.75	20.13	84.57	1.10
87.260	RRM	217	319.53	317.10	316.80	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.61	1.076	10.97	10.97	8.33	0	63.18	4.78	7.48	77.09	0.82	13.9	0.52	13.70	0.75	20.13	83.31	1.08
87.269	RRM	216	319.56	317.12	316.83	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.39	0.834	10.82	10.82	6.46	0	61.31	4.78	7.48	77.09	0.80	15.8	0.59	13.73	0.75	20.13	81.44	1.06
87.277	RRM	215	319.56	317.12	316.83	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.27	0.707	10.67	10.67	5.15	0	60.00	4.78	7.48	77.09	0.78	17.1	0.64	13.68	0.75	20.13	80.13	1.04
87.286	RRM	214	319.54	317.10	316.81	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.76	1.220	10.52	10.52	5.44	0	60.30	4.78	7.48	77.09	0.78	16.8	0.62	13.74	0.75	20.13	80.43	1.04
87.295	RRM	213	319.52	317.09	316.79	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.68	1.157	10.37	10.37	4.18	0	59.04	4.78	7.48	77.09	0.77	18.1	0.67	13.72	0.75	20.13	79.17	1.03
87.303	RRM	212	319.54	317.11	316.81	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.73	1.184	10.34	10.34	4.04	0	58.89	4.78	7.48	77.09	0.76	18.2	0.68	13.66	0.75	20.13	79.02	1.03
87.312	RRM	211	319.54	317.10	316.81	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.59	1.050	10.32	10.32	3.62	0	58.47	4.78	7.48	77.09	0.76	18.6	0.69	13.82	0.75	20.13	78.60	1.02
87.320	RRM	210	319.55	317.12	316.82	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.63	1.073	10.29	10.29	3.47	0	58.33	4.78	7.48	77.09	0.76	18.8	0.70	13.71	0.75	20.13	78.45	1.02
87.329	RRM	209	319.55	317.11	316.82	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.65	1.099	10.27	10.27	3.32	0	58.18	4.78	7.48	77.09	0.75	18.9	0.70	13.69	0.75	20.13	78.31	1.02
87.337	RRM	208	319.54	317.10	316.81	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.56	1.016	10.24	10.24	3.02	0	57.87	4.78	7.48	77.09	0.75	19.2	0.72	13.73	0.75	20.13	78.00	1.01
87.346	RRM	207	319.57	317.13	316.84	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.73	1.159	10.22	10.22	3.03	0	57.88	4.78	7.48	77.09	0.75	19.2	0.72	13.74	0.75	20.13	78.01	1.01
87.354	RRM	206	319.57	317.13	316.84	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.60	1.025	10.19	10.19	2.67	0	57.52	4.78	7.48	77.09	0.75	19.6	0.73	13.65	0.75	20.13	77.65	1.01
87.363	RRM	205	319.55	317.11	316.82	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.91	1.364	10.16	10.16	2.88	0	57.73	4.78	7.48	77.09	0.75	19.4	0.72	13.81	0.75	20.13	77.86	1.01
87.371	RRM	204	319.58	317.14	316.85	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.99	1.413	10.14	10.14	2.72	0	57.58	4.78	7.48	77.09	0.75	19.5	0.73	13.70	0.75	20.13	77.70	1.01
87.380	RRM	203	319.57	317.14	316.84	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.98	1.405	10.11	10.11	2.50	0	57.35	4.78	7.48	77.09	0.74	19.7	0.74	13.74	0.75	20.13	77.48	1.01
87.389	RRM	202	319.58	317.14	316.85	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.85	1.271	10.09	10.09	2.16	0	57.02	4.78	7.48	77.09	0.74	20.1	0.75	13.67	0.75	20.13	77.15	1.00
87.397	RRM	201	319.59	317.16	316.86	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.20	1.605	10.06	10.06	2.23	0	57.09	4.78	7.48	77.09	0.74	20.0	0.74	13.80	0.75	20.13	77.22	1.00
87.406	RRM	200	319.60	317.16	316.87	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.82	1.221	10.08	10.08	2.08	0	56.94	4.78	7.48	77.09	0.74	20.1	0.75	13.70	0.75	20.13	77.07	1.00
87.414	RRM	199	319.60	317.16	316.87	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.16	1.562	10.10	10.10	2.58	0	57.44	4.78	7.48	77.09	0.75	19.6	0.73	13.68	0.75	20.13	77.57	1.01
87.423	RRM	198	319.59	317.16	316.86	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	320.83	1.235	10.13	10.13	2.44	0	57.30	4.78	7.48	77.09	0.74	19.8	0.74	13.73	0.75	20.12	77.42	1.00
87.431	RRM	197	319.60	317.16	316.87	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.11	1.510	10.15	10.15	2.91	0	57.77	4.78	7.48	77.09	0.75	19.3	0.72	13.70	0.75	20.13	77.90	1.01
87.440	RRM	196	319.61	317.17	316.88	2.276	2.731	0.890	4.096	292	162	1.20	1.51	2.71	54.86	321.21	1.606	10.17	10.17	3.21	0	58.07	4.78	7.48	77.09	0.75	19.0	0.71	13.68	1.10	29.52	87.59	1.14
87.448	RA	195	319.61	317.17	316.92	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.25	1.642	10.19	10.19	3.02	0	48.02	4.78	7.00	72.07	0.67	24.1	1.00	13.74	1.10	26.59	74.61	1.04
87.457	RA	194	319.62	317.18	316.93	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.53	0.912	10.21	10.21	2.33	0	47.32	4.78	7.00	72.07	0.66	24.8	1.02	13.73	1.10	26.59	73.91	1.03
87.465	RA	193	319.63	317.19	316.94	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.65	1.022	10.24	10.24	2.56	0	47.59	4.78	7.00	72.07	0.66	24.5	1.01	13.73	1.10	26.59	74.17	1.03
87.474	RA	192	319.62	317.18	316.93	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.81	1.190	10.26	10.26	2.95	0	47.94	4.78	7.00	72.07	0.67	24.1	1.00	13.77	1.10	26.59	74.53	1.03
87.483	RA	191	319.63	317.19	316.94	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.10	1.472	10.28	10.28	3.45	0	48.48	4.78	7.00	72.07	0.67	23.6	0.98	13.71	1.10	26.59	75.07	1.04
87.491	RA	190	319.64	317.20	316.94	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.77	1.135	10.34	10.34	3.43	0	48.42	4.78	7.00	72.07	0.67	23.6	0.98	13.71	1.10	26.59	75.01	1.04
87.500	RA	189	319.62	317.19	316.93	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.25	1.622	10.41	10.41	4.70	0	49.70	4.78	7.00	72.07	0.69	22.4	0.93	13.79	1.10	26.59	76.28	1.06
87.508	RA	188	319.65	317.21	316.96	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.01	1.359	10.47	10.47	4.69	0	49.68	4.78	7.00	72.07	0.69	22.4	0.93	13.65	1.10	26.59	76.27	1.06
87.517	RA	187	319.65	317.21	316.96	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.60	0.954	10.54	10.54	4.27	0	49.27	4.78	7.00	72.07	0.68	22.8	0.94	13.66	1.10	26.59	75.85	1.05
87.525	RA	186	319.66	317.22	316.97	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.63	0.965	10.60	10.60	4.67	0	49.67	4.78	7.00	72.07	0.69	22.4	0.92	13.75	1.10	26.59	76.26	1.06
87.534	RA	185	319.66	317.23	316.97	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.98	1.318	10.66	10.66	5.94	0	50.94	4.78	7.00	72.07	0.71	21.1	0.87	13.76	1.10	26.59	77.53	1.08
87.542	RA	184	319.68	317.25	316.99	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.03	1.351	10.73	10.73	6.48	0	51.47	4.78	7.00	72.07	0.71	20.6	0.85	13.62	1.10	26.59	78.06	1.08
87.551	RA	183	319.67	317.23	316.98	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.20	1.535	10.79	10.79	7.46	0	52.46	4.78	7.00	72.07	0.73	19.6	0.81	13.80	1.10	26.59	79.05	1.10
87.559	RA	182	319.69	317.26	317.00	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.61	0.914	10.86	10.86	6.03	0	51.03	4.78	7.00	72.07	0.71	21.0	0.87	13.68	1.10	26.59	77.61	1.08
87.568	RA	181	319.69	317.25	317.00	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.70	1.012	10.92	10.92	6.72	0	51.72	4.78	7.00	72.07	0.72	20.4	0.84	13.74	1.10	26.59	78.31	1.09
87.577	RA	180	319.69	317.25	317.00	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.15	1.466	10.82	10.82	7.47	0	52.46	4.78	7.00	72.07	0.73	19.6	0.81	13.74	1.10	26.59	79.05	1.10
87.585	RA	179	319.69	317.25	317.00	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.66	0.967	10.72	10.72	5.39	0	50.38	4.78	7.00	72.07	0.70	21.7	0.90	13.80	1.10	26.59	76.97	1.07
87.594	RA	178	319.69	317.25	317.00	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.83	1.141	10.62	10.62	5.22	0	50.21	4.78	7.00	72.07	0.70	21.9	0.90	13.54	1.10	26.59	76.80	1.07
87.602	RA	177	319.70	317.27	317.01	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.15	1.443	10.52	10.52	5.20	0	50.20	4.78	7.00	72.07	0.70	21.9	0.91	13.80	1.10	26.59	76.78	1.07
87.611	RA	176	319.72	317.29	317.03	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.14	1.417	10.42	10.42	4.42	0	49.42	4.78	7.00	72.07	0.69	22.7	0.94	13.72	1.10	26.59	76.01	1.05
87.619	RA	175	319.73	317.29	317.03	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.05	1.320	10.32	10.32	3.55	0	48.55	4.78	7.00	72.07	0.67	23.5	0.97	13.70	1.10	26.59	75.14	1.04
87.628	RA	174	319.72	317.28	317.03	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.86	1.136	10.22	10.22	2.63	0	47.63	4.78	7.00	72.07	0.66	24.4	1.01	13.68	1.10	26.59	74.22	1.03
87.636	RA	173	319.73	317.29	317.04	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.00	1.268	10.12	10.12	2.11	0	47.11	4.78	7.00	72.07	0.65	25.0	1.03	13.78	1.10	26.59	73.69	1.02
87.645	RA	172	319.73	317.29	317.04	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.86	1.133	10.02	10.02	1.35	0	46.35	4.78	7.00	72.07	0.64	25.7	1.06	13.61	1.10	26.59	72.93	1.01
87.653	RA	171	319.71	317.28	317.02	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.28	1.568	9.92	9.92	0.85	0	45.84	4.78	7.00	72.07	0.64	26.2	1.08	13.78	1.10	26.59	72.43	1.00
87.662	RA	170	319.73	317.30	317.04	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.08	1.349	9.82	9.82	0.07	0	45.07	4.78	7.00	72.07	0.63	27.0	1.12	13.72	1.10	26.59	71.65	0.99
87.670	RA	169	319.75	317.31	317.06	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.25	1.497	9.95	9.95	1.05	0	46.05	4.78	7.00	72.07	0.64	26.0	1.08	13.67	1.10	26.59	72.64	1.01
87.679	RA	168	319.75	317.32	317.06	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.14	1.384	10.08	10.08	1.95	0	46.95	4.78	7.00	72.07	0.65	25.1	1.04	13.80	1.10	26.59	73.53	1.02
87.688	RA	167	319.75	317.31	317.06	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.25	1.497	10.21	10.21	3.02	0	48.01	4.78	7.00	72.07	0.67	24.1	1.00	13.64	1.10	26.59	74.60	1.04
87.696	RA	166	319.77	317.34	317.08	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.08	1.303	10.34	10.34	2.70	0	48.69	4.78	7.00	72.07	0.68	23.4	0.97	13.78	1.10	26.59	75.28	1.04
87.705	RA	165	319.78	317.34	317.09	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.30	1.520	10.48	10.48	5.03	0	50.02	4.78	7.00	72.07	0.69	22.0	0.91	13.69	1.10	26.59	76.61	1.06
87.713	RA	164	319.78	317.34	317.09	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.40	1.620	10.61	10.61	6.26	0	51.25	4.78	7.00	72.07	0.71	20.8	0.86	13.76	1.10	26.59	77.84	1.08
87.722	RA	163	319.77	317.33	317.08	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.30	1.525	10.74	10.74	7.03	0	52.03	4.78	7.00	72.07	0.72	20.0	0.83	13.79	1.10	26.59	78.61	1.09
87.730	RA	162	319.78	317.35	317.09	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.99	1.206	10.87	10.87	7.02	0	52.02	4.78	7.00	72.07	0.72	20.1	0.83	13.68	1.10	26.59	78.61	1.09
87.739	RA	161	319.78	317.34	317.09	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.15	1.369	11.00	11.00	8.47	0	53.46	4.78	7.00	72.07	0.74	18.6	0.77	13.72	1.10	26.59	80.05	1.11
87.747	RA	160	319.78	317.35	317.09	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.24	1.460	11.13	11.13	9.75	0	54.74	4.78	7.00	72.07	0.76	17.3	0.72	13.64	1.10	26.59	81.33	1.13
87.756	RA	159	319.79	317.35	317.10	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.15	1.359	11.26	11.26	10.29	0	55.28	4.78	7.00	72.07	0.77	16.8	0.69	13.82	1.10	26.59	81.87	1.14
87.764	RA	158	319.80	317.36	317.10	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.69	0.892	11.21	11.21	8.06	0	53.00	4.78	7.00	72.07	0.74	19.1	0.79	13.67	1.10	26.59	79.59	1.10
87.773	RA	157	319.80	317.36	317.11	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.90	1.097	11.16	11.16	8.54	0	53.54	4.78	7.00	72.07	0.74	18.5	0.77	13.65	1.10	26.59	80.13	1.11
87.782	RG	156	319.80	317.36	316.72	2.276	3.073	0.298	4.267	635	162	2.61	1.33	3.94	79.70	320.83	1.034	11.12	11.12	9.51	0	89.22	4.78	8.71	89.73	0.99	0.5	0.02	13.74	0.80	22.37	111.58	1.24
87.790	RG	155	319.79	317.36	316.72	2.276	3.073	0.298	4.267	635	162	2.61	1.33	3.94	79.70	320.99	1.199	11.07	11.07	9.86	0	89.56	4.78	8.71	89.73	1.00	0.2	0.01	13.75	0.80	22.37	111.93	1.25
87.799	RA	154	319.82	317.39	317.13	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.03	1.204	11.02	11.02	7.99	0	52.98	4.78	7.00	72.07	0.74	19.1	0.79	13.72	0.80	19.24	72.32	1.00
87.807	RE	153	319.82	317.39	317.23	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.34	1.516	10.97	10.97	10.33	0	51.68	4.78	6.82	70.22	0.74	18.5	0.68	13.65	0.80	22.43	74.11	1.06
87.816	RE	152	319.83	317.39	317.24	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.81	0.983	10.92	10.92	7.86	0	49.21	4.78	6.82	70.22	0.70	21.0	0.75	13.73	0.80	22.43	71.65	1.02
87.824	RE	151	319.83	317.39	317.24	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.82	0.995	10.87	10.87	7.56	0	48.91	4.78	6.82	70.22	0.70	21.3	0.76	13.74	0.80	22.43	71.34	1.02
87.833	RE	150	319.80	317.36	317.21	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.91	1.113	10.82	10.82	7.61	0	48.96	4.78	6.82	70.22	0.70	21.3	0.76	13.73	0.80	22.43	71.39	1.02
87.841	RE	149	319.83	317.40	317.24	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.83	0.999	10.77	10.77	6.86	0	48.21	4.78	6.82	70.22	0.69	22.0	0.78	13.75	0.80	22.43	70.65	1.01
87.850	RG	148	319.84	317.40	316.76	2.276	3.073	0.298	4.267	635	162	2.61	1.33	3.94	79.70	320.79	0.954	10.72	10.72	6.38	0	86.09	4.78	8.71	89.73	0.96	3.6	0.13	13.64	0.80	22.37	108.45	1.21
87.858	RG	147	319.84	317.41	316.77	2.276	3.073	0.298	4.267	635	162	2.61	1.33	3.94	79.70	319.97	0.123	10.67	10.67	3.59	0	83.30	4.78	8.71	89.73	0.93	6.4	0.23	13.77	0.80	22.37	105.66	1.18

SHOAL LAKE AQUEDUCT BUOYANCY ASSESSMENT - Boggy River Stretch (Mile 85.0 to 88.5) - Partial Safety Factor Method Using OHBDC (1991) Load and Resistance Factors

General Buoyancy Equation: [$\phi_{backfill}$ (backfill weight) + ϕ_{shear} (backfill shear resistance) + $\phi_{concrete}$ (weight of aqueduct) + $\phi_{ballast}$ (granular ballast weight)] ≥ α (buoyant force)

Backfill Notes
- measured or predicted unit weight
- submerged unit weights
- bulk unit weights (above crown only)

Backfill Shear Resistance:
Depth of Compacted Backfill Above Invert
- 0.0 = peat backfill (no shear resistance)
- 1.0 = clay layer (m) (depth contributing to shear resistance)

Concrete Unit Weight and Resistance Factor
- $\gamma_{Concrete}$ = 22.50 (kN/m³)
- Resistance Factor, ϕconcrete = 0.90

Backfill Resistance Factors
- Submerged Backfill, ϕsub. = 0.80
- Non-Submerged Crown Backfill, ϕbulk = 0.80
- Backfill Shear, ϕshear = 0.50
- Undrained Shear Strength = 12.0 (kPa)

Buoyancy Load Parameters
- γ_{water} = 9.81 (kN/m³)
- Buoyancy Load Factor, α = 1.05

Granular Ballast Parameters
- $\gamma_{Ballast\ bulk}$ = 14.40 (kN/m³)
- Resistance Factor, ϕbulk = 0.80 Non-Submerged ballast
- $\gamma_{Ballast\ Saturated}$ = 19.00 (kN/m³)
- Resistance Factor, ϕsub = 0.80 Submerged Ballast

Aqueduct Mileage	Aqueduct Segment Section Type	Aqueduct Segment No.	Aqueduct Survey Crown Elevation (m)	Invert Elevations Inside Aqueduct (m)	Invert Elevations Foundation Side (m)	Aqueduct Dimensions Inside Height (m)	Outside Height (m)	Area Below Haunch Grade (m²)	Maximum Exterior Width (m)	Invert Thickness (mm)	Crown Thickness (mm)	Concrete Area Invert (m²)	Arch (m²)	Total (m²)	Concrete Weight (kN/m)	Backfill Properties Average Backfill Elevation (m)	Depth on Centerline (m)	Saturated Unit Weight Crown (kN/m³)	Saturated Unit Weight Arch Leg (kN/m³)	Backfill Weight (kN/m)	Backfill Shear Resistance (kN/m)	Total Resistance R (kN/m)	Aqueduct Cross-section Area Inside (m²)	Aqueduct Cross-section Area Total (m²)	Buoyant Force B (kN/m)	Resisting Ratio R / B	Force to Resist (R / B<1) (kN/m)	Granular Ballast Minimum Depth Required (m)	Repair Length (m)	Design Depth (m)	Additional Resistance (kN/m)	New Total Resistance R (kN/m)	New Resisting Ratio R / B
87.867	RG	146	319.85	317.42	316.78	2.276	3.073	0.298	4.267	635	162	2.61	1.33	3.94	79.70	320.55	0.694	11.28	11.28	9.01	0	88.71	4.78	8.71	89.73	0.99	1.0	0.04	13.69	0.80	22.37	111.08	1.24
87.875	RA	145	319.87	317.43	317.18	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.78	0.914	11.89	11.89	12.00	0	56.99	4.78	7.00	72.07	0.79	15.1	0.62	13.69	0.80	19.34	76.33	1.06
87.884	RG	144	319.86	317.42	316.78	2.276	3.073	0.298	4.267	635	162	2.61	1.33	3.94	79.70	320.56	0.702	11.79	11.79	12.19	0	91.90	4.78	8.71	89.73	1.02	0.0	0.00	0.00	0.80	22.37	114.26	1.27
87.893	RE	143	319.86	317.42	317.27	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.46	0.600	11.69	11.69	10.89	0	52.25	4.78	6.82	70.22	0.74	18.0	0.64	13.71	0.80	22.43	74.68	1.06
87.901	RE	142	319.89	317.45	317.30	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.61	0.726	11.59	11.59	11.09	0	52.44	4.78	6.82	70.22	0.75	17.8	0.63	13.70	0.80	22.43	74.88	1.07
87.910	RE	141	319.88	317.44	317.29	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.42	0.535	11.49	11.49	9.37	0	50.72	4.78	6.82	70.22	0.72	19.5	0.70	13.81	0.80	22.43	73.15	1.04
87.915	R rd. xing	140	319.96	317.44	317.24	2.276	2.718	0.899	4.267	203	238	1.00	2.09	3.10	62.71	320.64	0.682	11.43	11.43	9.75	0	72.46	4.78	7.87	81.09	0.89	8.6	0.31	9.14	0.80	22.37	94.83	1.17
87.924	RE	139	319.88	317.45	317.29	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.96	1.078	11.33	11.33	11.28	0	52.63	4.78	6.82	70.22	0.75	17.6	0.69	13.65	0.80	22.43	75.06	1.07
87.932	RE	138	319.89	317.45	317.30	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.63	0.747	11.23	11.23	8.93	0	50.28	4.78	6.82	70.22	0.72	19.9	0.71	13.74	0.80	22.43	72.72	1.04
87.941	RE	137	319.89	317.45	317.30	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.60	0.706	11.13	11.13	8.12	0	49.47	4.78	6.82	70.22	0.70	20.7	0.74	13.74	0.80	22.43	71.91	1.02
87.950	RE	136	319.91	317.48	317.32	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.86	0.945	11.15	11.15	41.08	0	82.43	4.78	6.82	70.22	1.17	0.0	0.00	0.00	0.80	39.44	121.88	1.74
87.958	RE	135	319.90	317.47	317.31	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.19	1.289	11.17	11.17	54.38	0	95.73	4.78	6.82	70.22	1.36	0.0	0.00	0.00	0.80	39.44	135.17	1.93
87.967	RE	134	319.91	317.47	317.32	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.13	1.221	11.19	11.19	51.93	0	93.28	4.78	6.82	70.22	1.33	0.0	0.00	0.00	0.80	39.44	132.73	1.89
87.975	RE	133	319.91	317.47	317.32	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.25	1.339	11.21	11.21	56.62	0	97.97	4.78	6.82	70.22	1.40	0.0	0.00	0.00	0.00	0.00	97.97	1.40
87.984	RA	132	319.89	317.45	317.20	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.93	1.042	11.23	11.23	38.89	0	83.88	4.78	7.00	72.07	1.16	0.0	0.00	0.00	0.00	0.00	83.88	1.16
87.992	RE	131	319.87	317.43	317.28	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.79	0.925	11.25	11.25	41.00	0	82.35	4.78	6.82	70.22	1.17	0.0	0.00	0.00	0.00	0.00	82.35	1.17
88.001	RE	130	319.87	317.44	317.28	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.07	1.195	11.27	11.27	51.56	0	92.91	4.78	6.82	70.22	1.32	0.0	0.00	0.00	0.00	0.00	92.91	1.32
88.009	RE	129	319.88	317.45	317.29	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.71	0.825	11.29	11.29	37.41	0	78.76	4.78	6.82	70.22	1.12	0.0	0.00	0.00	0.00	0.00	78.76	1.12
88.018	RE	128	319.89	317.45	317.30	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.68	0.789	11.31	11.31	36.15	0	77.50	4.78	6.82	70.22	1.10	0.0	0.00	0.00	0.00	0.00	77.50	1.10
88.026	RE	127	319.90	317.46	317.31	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.90	0.996	11.33	11.33	44.30	0	85.65	4.78	6.82	70.22	1.22	0.0	0.00	0.00	0.00	0.00	85.65	1.22
88.035	RE	126	319.91	317.47	317.32	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.85	0.939	11.35	11.35	42.23	0	83.58	4.78	6.82	70.22	1.19	0.0	0.00	0.00	0.30	14.79	98.37	1.40
88.043	RE	125	319.89	317.45	317.30	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.01	1.118	11.37	11.37	49.33	0	90.68	4.78	6.82	70.22	1.29	0.0	0.00	0.00	0.30	14.79	105.47	1.50
88.052	RE	124	319.90	317.46	317.31	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.90	1.002	12.77	12.77	21.19	0	62.54	4.78	6.82	70.22	0.89	7.7	0.27	13.76	0.30	8.41	70.96	1.01
88.060	RE	123	319.91	317.47	317.32	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.10	1.188	14.16	14.16	33.92	0	75.27	4.78	6.82	70.22	1.07	0.0	0.00	0.00	0.30	8.41	83.69	1.19
88.069	RE	122	319.92	317.48	317.33	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.20	1.283	15.56	15.56	46.73	0	88.08	4.78	6.82	70.22	1.25	0.0	0.00	0.00	0.30	8.41	96.50	1.37
88.077	RE	121	319.93	317.49	317.34	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.93	1.002	15.26	15.26	39.02	0	80.38	4.78	6.82	70.22	1.14	0.0	0.00	0.00	0.30	8.41	88.79	1.26
88.086	RE	120	319.94	317.50	317.35	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.84	0.900	16.98	16.98	48.86	0	90.21	4.78	6.82	70.22	1.28	0.0	0.00	0.00	0.30	8.41	98.62	1.40
88.094	RE	119	319.95	317.51	317.36	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.10	1.153	18.70	18.70	68.29	0	109.64	4.78	6.82	70.22	1.56	0.0	0.00	0.00	0.30	8.41	118.06	1.68
88.103	RE	118	319.92	317.49	317.33	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.96	1.037	18.22	18.22	61.25	0	102.60	4.78	6.82	70.22	1.46	0.0	0.00	0.00	0.30	8.41	111.01	1.58
88.112	RE	117	319.93	317.50	317.34	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.94	1.007	17.74	17.74	56.95	0	98.30	4.78	6.82	70.22	1.40	0.0	0.00	0.00	0.30	8.41	106.71	1.52
88.120	RE	116	319.94	317.51	317.35	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.09	1.149	17.26	17.26	57.11	0	98.46	4.78	6.82	70.22	1.40	0.0	0.00	0.00	0.30	8.41	106.87	1.52
88.129	RE	115	319.95	317.52	317.36	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.04	1.085	16.78	16.78	51.91	0	93.26	4.78	6.82	70.22	1.33	0.0	0.00	0.00	0.30	8.41	101.67	1.45
88.137	RE	114	319.97	317.53	317.38	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.15	1.179	16.30	16.30	[illegible]	0	91.76	4.78	6.82	70.22	1.31	0.0	0.00	0.00	0.30	8.41	100.17	1.43
88.146	RE	113	319.98	317.54	317.39	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.06	1.083	16.18	16.18	[illegible]	0	88.77	4.78	6.82	70.22	1.26	0.0	0.00	0.00	0.30	8.41	97.19	1.38
88.154	RE	112	319.97	317.53	317.38	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.85	0.887	16.07	16.07	[illegible]	0	83.72	4.78	6.82	70.22	1.19	0.0	0.00	0.00	0.30	8.41	92.13	1.31
88.163	RE	111	319.97	317.53	317.38	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.79	0.827	15.95	15.95	[illegible]	0	81.68	4.78	6.82	70.22	1.16	0.0	0.00	0.00	0.30	8.41	90.09	1.28
88.171	RE	110	320.00	317.56	317.41	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.26	1.266	15.84	15.84	[illegible]	0	89.98	4.78	6.82	70.22	1.28	0.0	0.00	0.00	0.30	8.41	98.40	1.40
88.180	RE	109	319.98	317.55	317.39	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.39	1.407	15.72	15.72	[illegible]	0	91.91	4.78	6.82	70.22	1.31	0.0	0.00	0.00	0.30	8.41	100.33	1.43
88.188	RE	108	320.00	317.56	317.41	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.94	0.941	15.61	15.61	[illegible]	0	81.68	4.78	6.82	70.22	1.16	0.0	0.00	0.00	0.30	8.41	90.09	1.28
88.197	RE	107	320.01	317.57	317.42	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.23	1.217	15.49	15.49	[illegible]	0	86.25	4.78	6.82	70.22	1.23	0.0	0.00	0.00	0.30	8.41	94.67	1.35
88.205	RE	106	319.98	317.54	317.39	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.27	1.284	15.38	15.38	[illegible]	0	86.62	4.78	6.82	70.22	1.23	0.0	0.00	0.00	0.30	8.41	95.03	1.35
88.214	RE	105	319.99	317.56	317.40	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.98	0.985	15.26	15.26	[illegible]	0	80.11	4.78	6.82	70.22	1.14	0.0	0.00	0.00	0.30	8.41	88.52	1.26
88.222	RE	104	320.03	317.59	317.44	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.10	1.074	15.15	15.15	[illegible]	0	80.92	4.78	6.82	70.22	1.15	0.0	0.00	0.00	0.30	8.41	89.33	1.27
88.231	RE	103	320.01	317.58	317.42	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.91	0.898	15.03	15.03	[illegible]	0	76.92	4.78	6.82	70.22	1.10	0.0	0.00	0.00	0.30	8.41	85.33	1.22
88.239	RE	102	320.03	317.59	317.44	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.54	0.508	15.00	15.00	[illegible]	0	69.77	4.78	6.82	70.22	0.99	0.4	[illegible]	13.76	0.30	8.41	78.18	1.11
88.248	RE	101	320.04	317.60	317.45	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.13	1.083	14.97	14.97	[illegible]	0	79.75	4.78	6.82	70.22	1.14	0.0	0.00	0.00	0.30	8.41	88.17	1.26
88.257	RE	100	320.03	317.60	317.44	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.39	0.352	14.94	14.94	[illegible]	0	66.68	4.78	6.82	70.22	0.95	3.5	[illegible]	13.77	0.30	8.41	75.09	1.07
88.265	RE	99	320.03	317.59	317.44	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.02	0.991	14.90	14.90	[illegible]	12.00	89.67	4.78	6.82	70.22	1.28	0.0	0.00	0.00	0.30	8.41	98.08	1.40
88.274	RE	98	320.03	317.59	317.44	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.09	1.059	14.87	14.87	[illegible]	12.00	90.62	4.78	6.82	70.22	1.29	0.0	0.00	0.00	0.30	8.41	99.03	1.41
88.282	RE	97	320.05	317.61	317.46	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.18	1.132	14.84	14.84	[illegible]	12.00	91.64	4.78	6.82	70.22	1.31	0.0	0.00	0.00	0.30	8.41	100.05	1.42
88.291	RE	96	320.05	317.62	317.46	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.50	1.448	14.81	14.81	[illegible]	12.00	96.81	4.78	6.82	70.22	1.38	0.0	0.00	0.00	0.30	8.41	105.22	1.50
88.299	RE	95	320.06	317.62	317.47	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.13	1.077	14.78	14.78	[illegible]	12.00	90.22	4.78	6.82	70.22	1.28	0.0	0.00	0.00	0.30	8.41	98.63	1.40
88.308	RE	94	320.06	317.62	317.47	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	320.59	0.527	14.74	14.74	[illegible]	12.00	80.69	4.78	6.82	70.22	1.15	0.0	0.00	0.00	0.30	8.41	89.10	1.27
88.316	RE	93	320.06	317.63	317.47	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.30	1.234	14.71	14.71	[illegible]	12.00	92.38	4.78	6.82	70.22	1.32	0.0	0.00	0.00	0.30	8.41	100.79	1.44
88.325	RE	92	320.07	317.63	317.48	2.276	2.591	0.397	4.280	152	162	0.72	1.33	2.04	41.35	321.34	1.277	14.68	14.68	[illegible]	12.00	92.84	4.78	6.82	70.22	1.32	0.0	0.00	0.00	0.30	8.41	101.25	1.44
88.333	RA	91	320.08	317.64	317.38	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.23	1.155	14.48	14.48	[illegible]	12.00	87.22	4.78	7.00	72.07	1.21	0.0	0.00	0.00	0.30	[illegible]	94.47	1.31
88.342	RA	90	320.12	317.68	317.43	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.10	0.977	14.27	14.27	[illegible]	12.00	83.55	4.78	7.00	72.07	1.16	0.0	0.00	0.00	0.30	[illegible]	90.80	1.26
88.350	RA	89	320.10	317.66	317.41	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.50	1.400	14.07	14.07	[illegible]	12.00	87.64	4.78	7.00	72.07	1.22	0.0	0.00	0.00	0.30	[illegible]	94.89	1.32
88.359	RA	88	320.09	317.66	317.40	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.59	1.491	13.86	13.86	[illegible]	12.00	87.24	4.78	7.00	72.07	1.21	0.0	0.00	0.00	0.30	[illegible]	94.49	1.31
88.367	RA	87	320.04	317.60	317.34	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.36	1.322	13.66	13.66	[illegible]	12.00	83.80	4.78	7.00	72.07	1.16	0.0	0.00	0.00	0.30	[illegible]	91.05	1.26
88.376	RA	86	320.04	317.60	317.35	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.47	1.424	13.45	13.45	[illegible]	12.00	83.47	4.78	7.00	72.07	1.16	0.0	0.00	0.00	0.30	[illegible]	90.72	1.26
88.384	RA	85	320.06	317.62	317.37	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.70	1.638	13.25	13.25	[illegible]	12.00	84.15	4.78	7.00	72.07	1.17	0.0	0.00	0.00	0.30	[illegible]	91.40	1.27
88.393	RA	84	320.07	317.63	317.38	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.61	1.535	13.04	13.04	[illegible]	12.00	81.54	4.78	7.00	72.07	1.13	0.0	0.00	0.00	0.30	[illegible]	88.79	1.23
88.401	RA	83	320.08	317.65	317.39	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.25	1.165	13.61	13.61	[illegible]	12.00	81.73	4.78	7.00	72.07	1.13	0.0	0.00	0.00	0.30	[illegible]	88.98	1.23
88.410	RA	82	320.10	317.66	317.41	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.00	0.903	11.76	11.76	[illegible]	12.00	68.18	4.78	7.00	72.07	0.95	3.9	[illegible]	13.74	0.30	[illegible]	75.43	1.05
88.419	RA	81	320.11	317.67	317.42	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.91	1.803	12.09	12.09	[illegible]	12.00	76.16	4.78	7.00	72.07	1.06	0.0	0.00	0.00	0.30	[illegible]	83.41	1.16
88.427	RA	80	320.09	317.65	317.40	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	320.27	0.184	12.43	12.43	[illegible]	12.00	66.45	4.78	7.00	72.07	0.92	5.6	[illegible]	13.74	0.30	[illegible]	73.70	1.02
88.436	RA	79	320.10	317.66	317.41	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.01	0.915	12.76	12.76	[illegible]	12.00	74.01	4.78	7.00	72.07	1.03	0.0	0.00	0.00	0.30	[illegible]	81.26	1.13
88.444	RA	78	320.11	317.67	317.42	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.61	1.504	13.09	13.09	[illegible]	12.00	81.64	4.78	7.00	72.07	1.13	0.0	0.00	0.00	0.30	[illegible]	88.89	1.23
88.453	RA	77	320.12	317.68	317.43	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.45	1.327	13.43	13.43	[illegible]	12.00	82.25	4.78	7.00	72.07	1.14	0.0	0.00	0.00	0.30	[illegible]	89.50	1.24
88.461	RA	76	320.11	317.67	317.42	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.42	1.308	13.76	13.76	[illegible]	12.00	84.37	4.78	7.00	72.07	1.17	0.0	0.00	0.00	0.30	[illegible]	91.62	1.27
88.470	RA	75	320.12	317.69	317.43	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.43	1.305	13.32	13.32	[illegible]	12.00	81.27	4.78	7.00	72.07	1.13	0.0	0.00	0.00	0.30	[illegible]	88.52	1.23
88.478	RA	74	320.14	317.70	317.44	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.26	1.130	12.87	12.87	[illegible]	12.00	76.52	4.78	7.00	72.07	1.06	0.0	0.00	0.00	0.30	[illegible]	83.77	1.16
88.487	RA	73	320.11	317.67	317.42	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.45	1.342	12.43	12.43	[illegible]	12.00	75.41	4.78	7.00	72.07	1.05	0.0	0.00	0.00	0.30	[illegible]	82.67	1.15
88.495	RA	72	320.13	317.69	317.44	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.33	1.198	12.52	12.52	[illegible]	12.00	74.90	4.78	7.00	72.07	1.04	0.0	0.00	0.00	0.30	[illegible]	82.15	1.14
88.504	RA	71	320.14	317.71	317.45	2.276	2.692	0.903	3.689	254	162	0.90	1.33	2.22	45.00	321.59	1.449	12.61	12.61	[illegible]	12.00	77.56	4.78	7.00	72.07	1.08	0.0	0.00	0.00	0.00	[illegible]	77.56	1.08

APPENDIX 5-2. Simulation model output for the 'BP' aqueduct section.

Monte Carlo Simulation Summary for the BP Aqueduct Section

Summary Information	
Workbook Name	eliability analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200

Output		Statistics						
Name	Cell	Minimum	Mean	Maximum	x1	p1	x2	p2
Crown Backfill Weight	B26	0.505	13.818	75.594	0.757	0.011%	68.490	99.989%
Arch Leg Backfill Weight	B27	1.229	12.893	41.845	1.376	0.011%	41.798	99.989%
Shear Resistance	B28	0.000	0.000	0.000	0.000	0.011%	0.000	99.989%
Aqueduct Self Weight	B29	38.543	49.730	61.374	38.904	0.011%	61.181	99.989%
Total Resisting Force	B30	44.621	76.441	171.892	45.388	0.011%	163.124	99.989%
Resisting Ratio	B32	0.681	1.151	2.559	0.695	0.011%	2.452	99.989%
Minimum ballast depth	B37	0.000	0.061	0.643	4.055	0.011%	0.607	99.989%
New Total Resisting Force	B38	64.325	78.464	171.892	64.348	0.011%	163.124	99.989%
New Resisting Ratio	B39	1.000	1.182	2.559	1.000	0.011%	2.452	99.989%

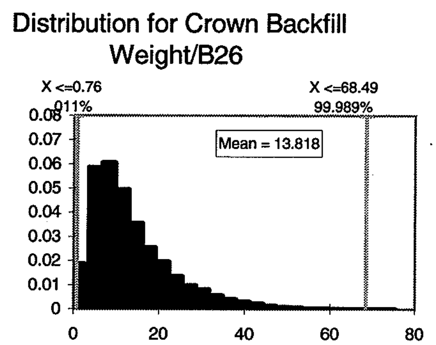
Input		Statistics						
Name	Cell	Minimum	Mean	Maximum	x1	p1	x2	p2
Saturated backfill unit wt. =	B8	10.109	12.949	19.999	10.145	0.011%	19.987	99.989%
Backfill depth =	B9	0.155	1.084	1.997	0.171	0.011%	1.976	99.989%
Concrete unit weight =	B12	21.473	23.351	24.893	21.904	0.011%	24.781	99.989%
Concrete area =	B13	1.710	2.130	2.549	1.716	0.011%	2.545	99.989%
Total section area =	B14	6.557	6.765	6.974	6.559	0.011%	6.972	99.989%
Saturated ballast unit weight =	B15	16.252	18.025	19.889	16.377	0.011%	19.671	99.989%

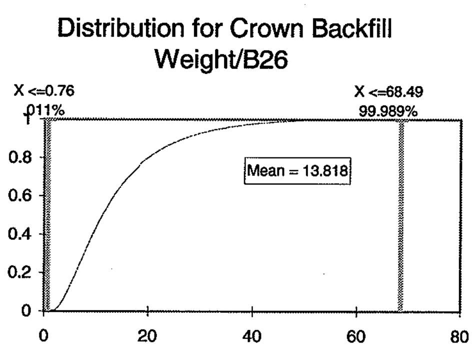
Monte Carlo Simulation Summary for the BP Aqueduct Section Summary of Input Statistics						
Simulation	Saturated Backfill Unit Weight (kN/m^3)	Crown Backfill Depth (m)	Concrete Unit Weight (kN/m^3)	Concrete Area (m^2)	Total Section Area (m^2)	Saturated Ballast Unit Weight (kN/m^3)
Minimum	10.109	0.155	21.473	1.710	6.557	16.252
Maximum	19.999	1.997	24.893	2.549	6.974	19.889
Mean	12.949	1.084	23.351	2.130	6.765	18.025
Standard Deviation	1.985	0.268	0.395	0.172	0.086	0.446
Coeff. Of Variation	0.153	0.247	0.017	0.081	0.013	0.025
Variance	3.942	0.072	0.156	0.030	0.007	0.199
Skewness	1.195	-0.002	-0.002	0.009	0.012	0.000
Kurtosis	4.055	2.932	3.000	2.400	2.400	2.996
Number of Errors	0.000	0.000	0.000	0.000	0.000	0.000
Mode	11.414	1.074	23.152	2.127	6.692	17.589
5%	10.708	0.642	22.702	1.842	6.622	17.291
10%	10.930	0.740	22.845	1.897	6.649	17.453
15%	11.118	0.805	22.942	1.939	6.670	17.562
20%	11.293	0.858	23.019	1.975	6.688	17.649
25%	11.465	0.903	23.085	2.006	6.703	17.724
30%	11.638	0.943	23.144	2.035	6.717	17.791
35%	11.816	0.980	23.199	2.061	6.730	4.055
40%	12.003	1.016	23.251	2.085	6.742	17.912
45%	12.200	1.050	23.301	2.107	6.754	17.969
50%	12.413	1.084	23.351	2.129	6.764	18.025
55%	12.645	1.118	23.401	2.151	6.775	18.081
60%	12.902	1.152	23.451	2.174	6.787	18.138
65%	13.191	1.187	23.503	2.198	6.799	18.197
70%	13.524	1.225	23.558	2.224	6.812	18.259
75%	13.917	1.265	23.617	2.253	6.826	18.326
80%	14.395	1.310	23.683	2.284	6.842	18.401
85%	15.006	1.362	23.760	2.320	6.860	18.488
90%	15.846	1.428	23.857	2.363	6.881	18.597
95%	17.160	1.526	24.000	2.418	6.908	18.759
Filter Minimum						
Filter Maximum						
Type (1 or 2)						
# Values Filtered	0	0	0	0	0	0
Scenario #1						
Scenario #2						
Scenario #3						
Target #1 (Value)	10.145	0.171	21.904	1.716	6.559	16.377
Target #1 (Perc%)	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	19.987	1.976	24.781	2.545	6.972	19.671
Target #2 (Perc%)	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						
Target #3 (Perc%)						

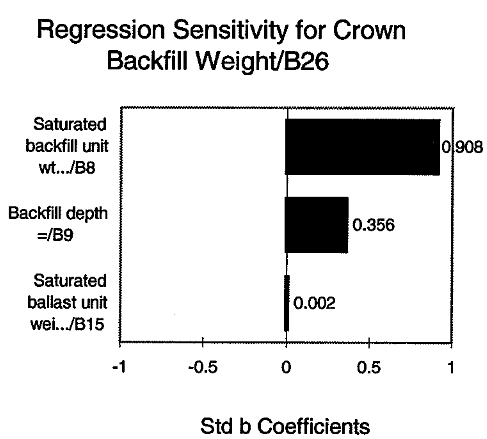
Monte Carlo Simulation Summary for the BP Aqueduct Section Summary of Output Statistics									
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.505	1.229	n/a	38.543	44.621	0.681	0.000	64.325	1.000
Maximum	75.594	41.845	n/a	61.374	171.892	2.559	0.643	171.892	2.559
Mean	13.818	12.893	n/a	49.730	76.441	1.151	0.061	78.464	1.182
Standard Deviation	9.603	8.154	n/a	4.103	17.820	0.267	0.111	15.927	0.238
Coeff. Of Variation	0.695	0.632	n/a	0.083	0.233	0.231	1.828	0.203	0.202
Variance	92.217	66.495	n/a	16.834	317.537	0.071	0.012	253.664	0.057
Skewness	1.496	1.195	n/a	0.029	1.185	1.206	1.923	1.624	1.643
Kurtosis	5.627	4.055	n/a	2.455	4.368	4.396	5.993	5.438	5.479
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	7.752	6.586	n/a	43.355	79.783	0.849	0.000	79.783	1.000
5%	3.442	3.688	n/a	42.918	55.273	0.839	0.000	65.255	1.000
10%	4.440	4.601	n/a	44.239	58.014	0.879	0.000	65.679	1.000
15%	5.289	5.371	n/a	45.234	60.109	0.908	0.000	66.021	1.000
20%	6.126	6.091	n/a	46.066	61.915	0.934	0.000	66.320	1.000
25%	6.878	6.797	n/a	46.800	63.570	0.959	0.000	66.616	1.000
30%	7.691	7.508	n/a	47.456	65.243	0.982	0.000	67.000	1.000
35%	8.473	8.240	n/a	48.071	66.827	4.055	0.000	67.506	1.006
40%	9.325	9.005	n/a	48.627	68.489	1.031	0.000	68.489	1.031
45%	10.221	9.817	n/a	49.157	70.277	1.058	0.000	70.277	1.058
50%	11.186	10.691	n/a	49.683	72.042	1.085	0.000	72.042	1.085
55%	12.254	11.644	n/a	50.203	74.184	1.116	0.000	74.184	1.116
60%	13.332	12.699	n/a	50.777	76.392	1.149	0.000	76.392	1.149
65%	14.661	13.888	n/a	51.362	78.862	1.186	0.000	78.862	1.186
70%	16.198	15.255	n/a	51.967	81.698	1.229	0.036	81.698	1.229
75%	18.017	16.865	n/a	52.642	85.048	1.280	0.082	85.048	1.280
80%	20.262	18.829	n/a	53.409	89.090	1.340	0.131	89.090	1.340
85%	23.102	21.338	n/a	54.268	94.580	1.422	0.183	94.580	1.422
90%	27.082	24.788	n/a	55.271	101.631	1.530	0.240	101.631	1.530
95%	33.478	30.185	n/a	56.631	112.959	1.704	0.319	112.959	1.704
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.757	1.376	0.000	38.904	45.388	0.695	0.000	64.348	1.000
Target #1 (Perc%)	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	68.490	41.798	0.000	61.181	163.124	2.452	0.607	163.124	2.452
Target #2 (Perc%)	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.643		
Target #3 (Perc%)						33.560%	100.000%		

Monte Carlo Simulation Summary for the BP Aqueduct Section

Simulation Results for Crown Backfill Weight







Summary Information	
Workbook Name	ty analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200

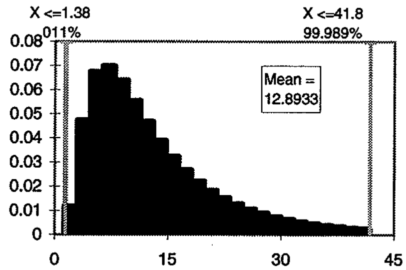
Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.505	5%	3.44
Maximum	75.594	10%	4.44
Mean	13.818	15%	5.29
Std Dev	9.603	20%	6.13
Variance	92.217	25%	6.88
Skewness	1.496	30%	7.69
Kurtosis	4.055	35%	8.47
Median	11.186	40%	9.32
Mode	7.752	45%	10.22
Left X	0.757	50%	11.19
Left P	0.01%	55%	12.25
Right X	68.49	60%	13.33
Right P	99.99%	65%	14.66
Diff X	67.73	70%	16.20
Diff P	99.98%	75%	18.02
#Errors	0	80%	20.26
Filter Min		85%	23.10
Filter Max		90%	27.08
#Filtered	0	95%	33.48

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.908	0.921
#2	Backfill depth	0.356	0.351
#3	Saturated ballast unit weight	0.002	0.006
#4	Concrete unit weight	0.000	0.001
#5	Concrete area	0.000	0.004
#6	Total section area	0.000	0.002
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

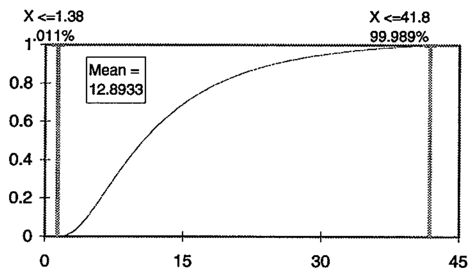
Monte Carlo Simulation Summary for the BP Aqueduct Section

Simulation Results for Arch Leg Backfill Weight

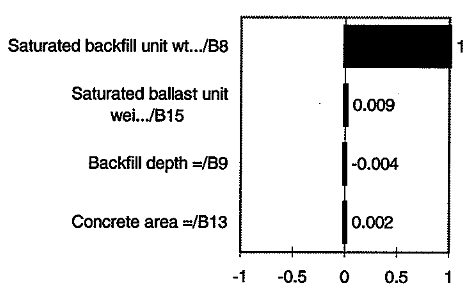
Distribution for Arch Leg Backfill Weight/B27



Distribution for Arch Leg Backfill Weight/B27



Correlations for Arch Leg Backfill Weight/B27



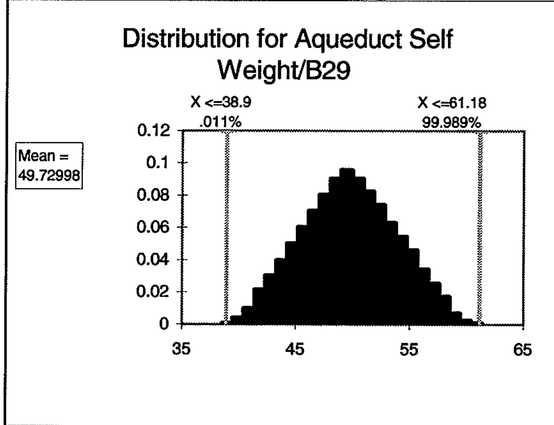
Summary Information	
Workbook Name	ty analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	1.229	5%	3.69
Maximum	41.845	10%	4.60
Mean	12.893	15%	5.37
Std Dev	8.154	20%	6.09
Variance	66.495	25%	6.80
Skewness	1.195	30%	7.51
Kurtosis	4.055	35%	8.24
Median	10.691	40%	9.01
Mode	6.586	45%	9.82
Left X	1.376	50%	10.69
Left P	0.01%	55%	11.64
Right X	41.80	60%	12.70
Right P	99.99%	65%	13.89
Diff X	40.42	70%	15.25
Diff P	99.98%	75%	16.87
#Errors	0	80%	18.83
Filter Min		85%	21.34
Filter Max		90%	24.79
#Filtered	0	95%	30.19

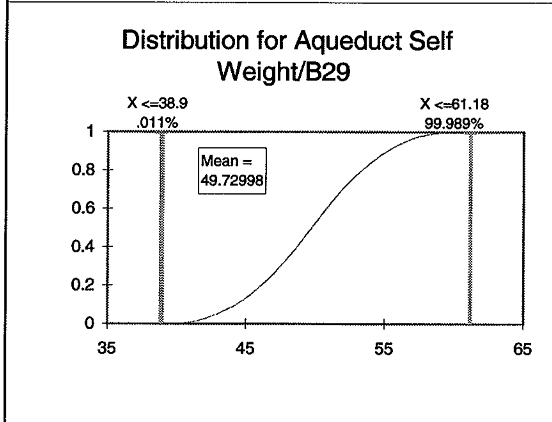
Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	1.000	1.000
#2	Concrete area	0.000	0.002
#3	Total section area	0.000	0.000
#4	Saturated ballast unit weight	0.000	0.009
#5	Backfill depth	0.000	-0.004
#6	Concrete unit weight	0.000	0.000
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the BP Aqueduct Section

Simulation Results for Aqueduct Weight

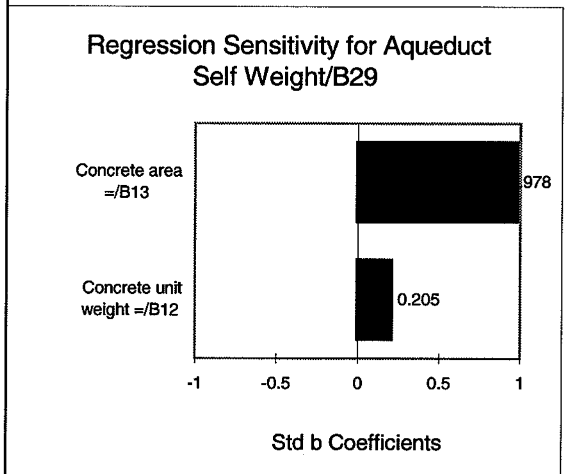


Summary Information	
Workbook Name	ity analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200



Summary Statistics

Statistic	Value	%tile	Value
Minimum	38.543	5%	42.92
Maximum	61.374	10%	44.24
Mean	49.730	15%	45.23
Std Dev	4.103	20%	46.07
Variance	16.834	25%	46.80
Skewness	0.029	30%	47.46
Kurtosis	4.055	35%	48.07
Median	49.683	40%	48.63
Mode	43.355	45%	49.16
Left X	38.904	50%	49.68
Left P	0.01%	55%	50.20
Right X	61.18	60%	50.78
Right P	99.99%	65%	51.36
Diff X	22.28	70%	51.97
Diff P	99.98%	75%	52.64
#Errors	0	80%	53.41
Filter Min		85%	54.27
Filter Max		90%	55.27
#Filtered	0	95%	56.63

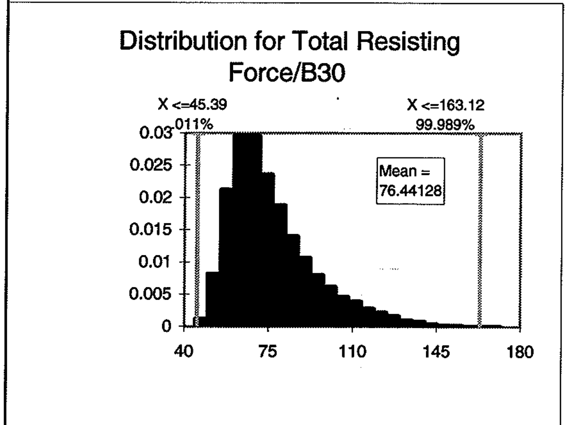


Sensitivity

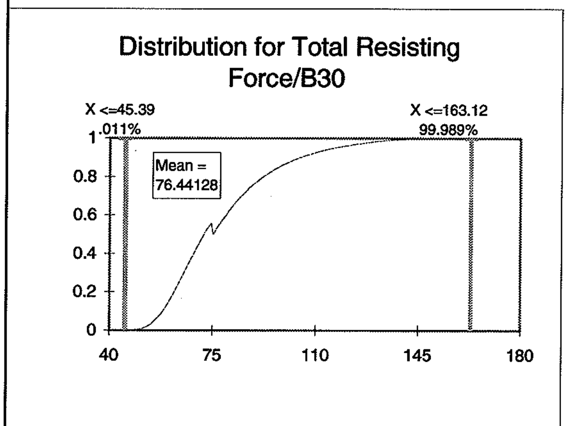
Rank	Name	Regr	Corr
#1	Concrete area	0.978	0.979
#2	Concrete unit weight	0.205	0.194
#3	Saturated backfill unit wt.	0.000	0.002
#4	Backfill depth	0.000	0.005
#5	Total section area	0.000	0.736
#6	Saturated ballast unit weight	0.000	-0.007
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the BP Aqueduct Section

Simulation Results for Total Resisting Force

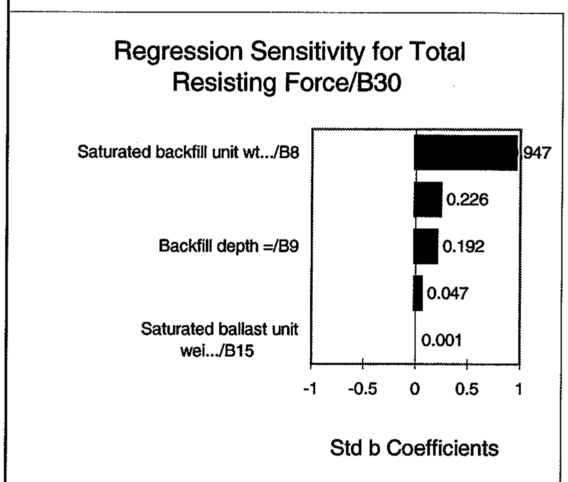


Summary Information	
Workbook Name	ty analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200



Summary Statistics

Statistic	Value	%tile	Value
Minimum	44.621	5%	55.27
Maximum	171.892	10%	58.01
Mean	76.441	15%	60.11
Std Dev	17.820	20%	61.91
Variance	317.537	25%	63.57
Skewness	1.185	30%	65.24
Kurtosis	4.055	35%	66.83
Median	72.042	40%	68.49
Mode	79.783	45%	70.28
Left X	45.388	50%	72.04
Left P	0.01%	55%	74.18
Right X	163.12	60%	76.39
Right P	99.99%	65%	78.86
Diff X	117.74	70%	81.70
Diff P	99.98%	75%	85.05
#Errors	0	80%	89.09
Filter Min		85%	94.58
Filter Max		90%	101.63
#Filtered	0	95%	112.96



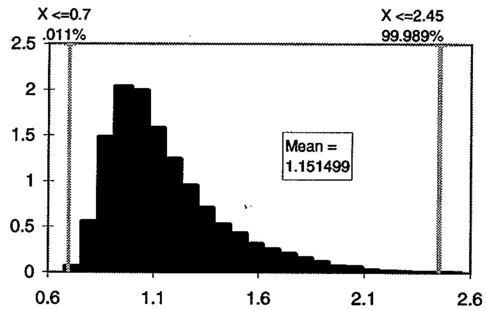
Sensitivity

Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.947	0.930
#2	Concrete area	0.226	0.276
#3	Backfill depth	0.192	0.173
#4	Concrete unit weight	0.047	0.057
#5	Saturated ballast unit weight	0.001	0.004
#6	Total section area	0.000	0.208
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

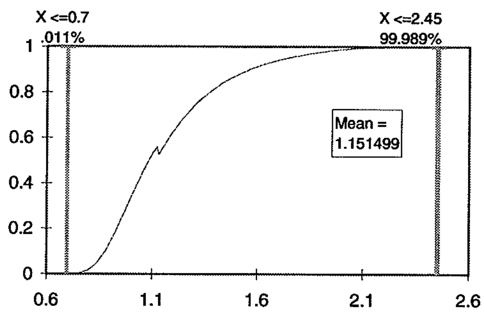
Monte Carlo Simulation Summary for the BP Aqueduct Section

Simulation Results for Resisting Ratio

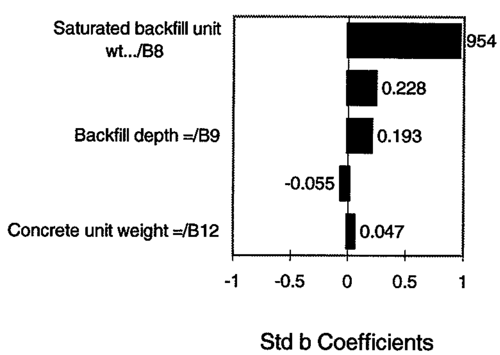
Distribution for Resisting Ratio/B32



Distribution for Resisting Ratio/B32



Regression Sensitivity for Resisting Ratio/B32



Summary Information	
Workbook Name	analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200

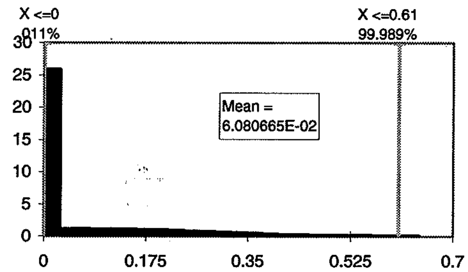
Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.681	5%	0.84
Maximum	2.559	10%	0.88
Mean	1.151	15%	0.91
Std Dev	0.267	20%	0.93
Variance	0.071	25%	0.96
Skewness	1.206	30%	0.98
Kurtosis	4.055	35%	1.01
Median	1.085	40%	1.03
Mode	0.849	45%	1.06
Left X	0.695	50%	1.08
Left P	0.01%	55%	1.12
Right X	2.45	60%	1.15
Right P	99.99%	65%	1.19
Diff X	1.76	70%	1.23
Diff P	99.98%	75%	1.28
#Errors	0	80%	1.34
Filter Min		85%	1.42
Filter Max		90%	1.53
#Filtered	0	95%	1.70

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.954	0.942
#2	Concrete area	0.228	0.234
#3	Backfill depth	0.193	0.175
#4	Total section area	-0.055	0.151
#5	Concrete unit weight	0.047	0.058
#6	Saturated ballast unit weight	0.000	0.005
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

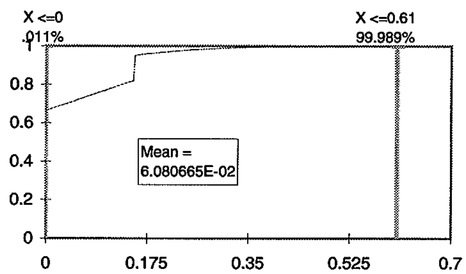
Monte Carlo Simulation Summary for the BP Aqueduct Section

Simulation Results for Minimum Ballast Depth

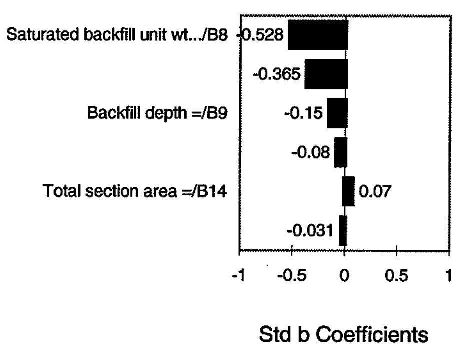
Distribution for Minimum ballast depth/B37



Distribution for Minimum ballast depth/B37



Regression Sensitivity for Minimum ballast depth/B37



Summary Information	
Workbook Name	ty analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200

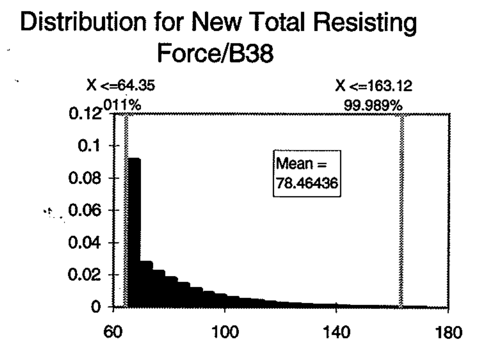
Summary Statistics

Statistic	Value	%tile	Value
Minimum	0.000	5%	0.00
Maximum	0.643	10%	0.00
Mean	0.061	15%	0.00
Std Dev	0.111	20%	0.00
Variance	0.012	25%	0.00
Skewness	1.923	30%	0.00
Kurtosis	4.055	35%	0.00
Median	0.000	40%	0.00
Mode	0.000	45%	0.00
Left X	0.000	50%	0.00
Left P	0.01%	55%	0.00
Right X	0.61	60%	0.00
Right P	99.99%	65%	0.00
Diff X	0.61	70%	0.04
Diff P	99.98%	75%	0.08
#Errors	0	80%	0.13
Filter Min		85%	0.18
Filter Max		90%	0.24
#Filtered	0	95%	0.32

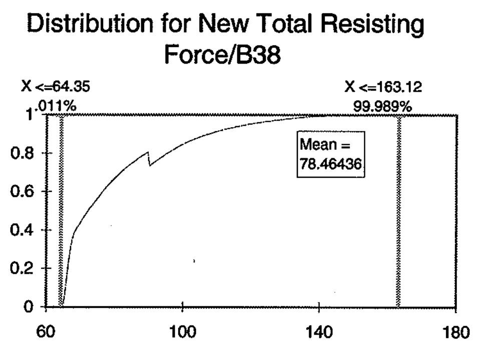
Sensitivity

Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	-0.528	-0.772
#2	Concrete area	-0.365	-0.266
#3	Backfill depth	-0.150	-0.149
#4	Concrete unit weight	-0.080	-0.069
#5	Total section area	0.070	-0.175
#6	Saturated ballast unit weight	-0.031	-0.010
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the BP Aqueduct Section
Simulation Results for New Total Resisting Force
(includes minimum ballast depth)

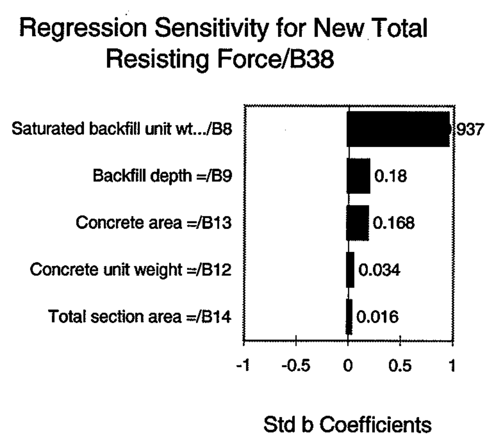


Summary Information	
Workbook Name	ty analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200



Summary Statistics

Statistic	Value	%tile	Value
Minimum	64.325	5%	65.25
Maximum	171.892	10%	65.68
Mean	78.464	15%	66.02
Std Dev	15.927	20%	66.32
Variance	253.664	25%	66.62
Skewness	4.055	30%	67.00
Kurtosis	5.438	35%	67.51
Median	72.042	40%	68.49
Mode	79.783	45%	70.28
Left X	64.348	50%	72.04
Left P	0.01%	55%	74.18
Right X	163.12	60%	76.39
Right P	99.99%	65%	78.86
Diff X	98.78	70%	81.70
Diff P	99.98%	75%	85.05
#Errors	0	80%	89.09
Filter Min		85%	94.58
Filter Max		90%	101.63
#Filtered	0	95%	112.96

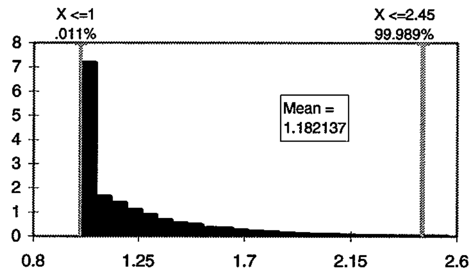


Sensitivity

Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.937	0.881
#2	Backfill depth	0.180	0.158
#3	Concrete area	0.168	0.313
#4	Concrete unit weight	0.034	0.042
#5	Total section area	0.016	0.292
#6	Saturated ballast unit weight	0.000	0.003
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

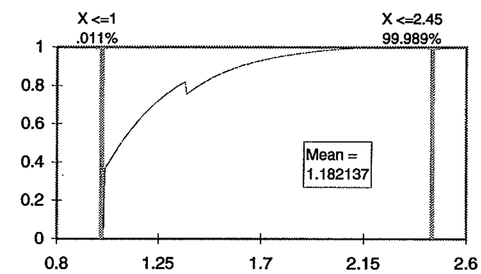
Monte Carlo Simulation Summary for the BP Aqueduct Section
Simulation Results for New Resisting Ratio
(includes minimum ballast depth)

Distribution for New Resisting Ratio/B39



Summary Information	
Workbook Name	ty analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200

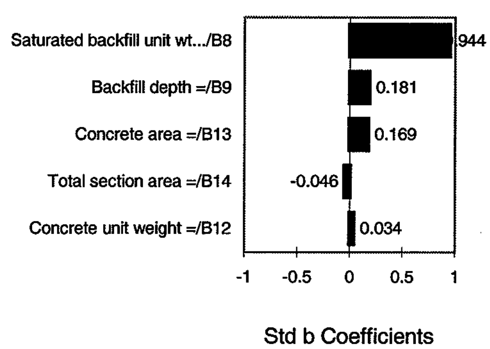
Distribution for New Resisting Ratio/B39



Summary Statistics

Statistic	Value	%tile	Value
Minimum	1.000	5%	1.00
Maximum	2.559	10%	1.00
Mean	1.182	15%	1.00
Std Dev	0.238	20%	1.00
Variance	0.057	25%	1.00
Skewness	4.055	30%	1.00
Kurtosis	5.479	35%	1.01
Median	1.085	40%	1.03
Mode	1.000	45%	1.06
Left X	1.000	50%	1.08
Left P	0.01%	55%	1.12
Right X	2.45	60%	1.15
Right P	99.99%	65%	1.19
Diff X	1.45	70%	1.23
Diff P	99.98%	75%	1.28
#Errors	0	80%	1.34
Filter Min		85%	1.42
Filter Max		90%	1.53
#Filtered	0	95%	1.70

Regression Sensitivity for New Resisting Ratio/B39



Sensitivity

Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.944	0.929
#2	Backfill depth	0.181	0.168
#3	Concrete area	0.169	0.197
#4	Total section area	-0.046	0.125
#5	Concrete unit weight	0.034	0.048
#6	Saturated ballast unit weight	0.000	0.005
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

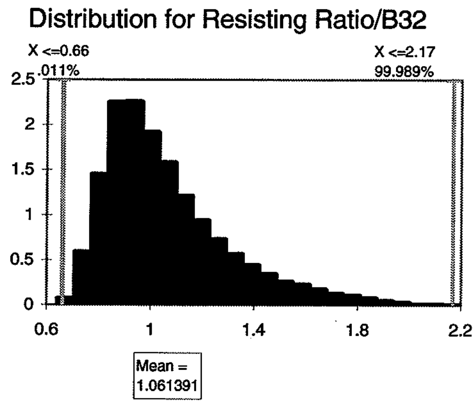
APPENDIX 5-3. Simulation model output statistics and simulation summary for the resisting ratio, all aqueduct section

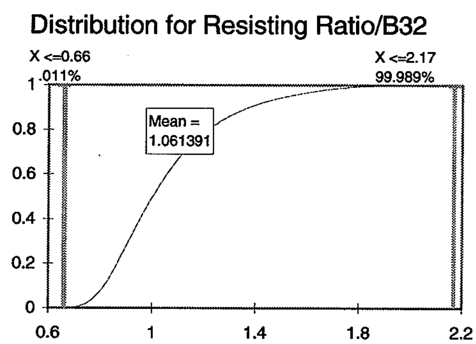
Monte Carlo Simulation Summary for the BO Aqueduct Section
Summary of Output Statistics

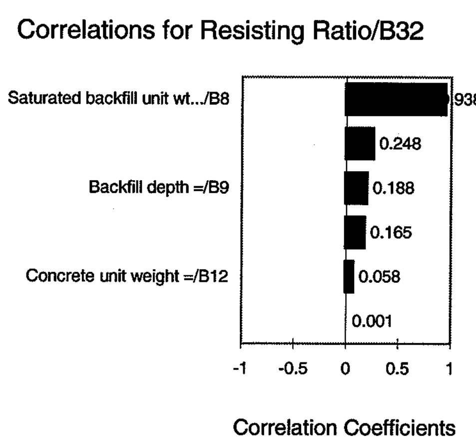
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.652	1.041	n/a	34.769	40.818	0.643	0.000	62.608	1.000
Maximum	63.092	34.840	n/a	56.127	145.404	2.198	0.776	145.404	2.198
Mean	12.457	10.737	n/a	45.380	68.573	1.061	0.126	72.359	1.120
Standard Deviation	8.725	6.790	n/a	3.759	15.617	0.240	0.168	12.569	0.193
Coeff. Of Variation	0.700	0.632	n/a	0.083	0.228	0.226	1.333	0.174	0.172
Variance	76.120	46.109	n/a	14.132	243.883	0.057	0.028	157.990	0.037
Skewness	1.509	1.195	n/a	0.035	1.191	1.214	1.141	1.961	1.987
Kurtosis	5.585	4.055	n/a	2.455	4.389	4.426	3.195	6.706	6.782
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	4.129	5.367	n/a	37.466	48.304	0.812	0.000	64.126	1.000
5%	3.113	3.071	n/a	39.147	49.957	0.779	0.000	63.393	1.000
10%	4.016	3.831	n/a	40.359	52.465	0.816	0.000	63.737	1.000
15%	4.768	4.473	n/a	41.236	54.320	0.843	0.000	64.004	1.000
20%	5.473	5.072	n/a	42.014	55.876	0.866	0.000	64.236	1.000
25%	6.173	5.660	n/a	42.671	57.336	0.888	0.000	64.452	1.000
30%	6.867	6.252	n/a	43.299	58.786	0.910	0.000	64.644	1.000
35%	7.584	6.861	n/a	43.862	60.172	0.932	0.000	64.856	1.000
40%	8.334	7.499	n/a	44.372	61.611	0.953	0.000	65.104	1.000
45%	9.160	8.175	n/a	44.868	63.163	0.977	0.000	65.403	1.000
50%	10.026	8.903	n/a	45.343	64.765	1.002	0.000	65.803	1.002
55%	10.988	9.696	n/a	45.823	66.599	1.029	0.049	66.599	1.029
60%	12.039	10.575	n/a	46.326	68.520	1.059	0.101	68.520	1.059
65%	13.192	11.565	n/a	46.874	70.702	1.092	0.147	70.702	1.092
70%	14.582	12.703	n/a	47.443	73.194	1.132	0.194	73.194	1.132
75%	16.153	14.044	n/a	48.060	76.106	1.177	0.240	76.106	1.177
80%	18.211	15.680	n/a	48.742	79.747	1.231	0.287	79.747	1.231
85%	20.799	17.770	n/a	49.520	84.299	1.303	0.337	84.299	1.303
90%	24.575	20.641	n/a	50.428	90.618	1.402	0.393	90.618	1.402
95%	30.492	25.134	n/a	51.705	100.871	1.559	0.475	100.871	1.559
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	n/a	0	0	0	0	0	0
Target #1 (Value)	0.813	1.159	n/a	35.771	41.858	0.661	0.000	62.629	1.000
Target #1 (Perc%)	0.013%	0.011%	n/a	0.013%	0.013%	0.013%	50.433%	0.013%	49.567%
Target #2 (Value)	61.650	34.804	n/a	55.736	138.451	2.169	0.762	138.451	2.169
Target #2 (Perc%)	99.990%	99.989%	n/a	99.990%	99.990%	99.990%	99.990%	99.990%	99.990%
Target #3 (Value)						1.000	0.776		
Target #3 (Perc%)						49.567%	100.000%		

Monte Carlo Simulation Summary for the BO Aqueduct Section

Simulation Results for Resisting Ratio







Summary Information	
Workbook Name	ty analysis BO section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:27
Simulation Stop Time	4/29/2002 17:28
Simulation Duration	00:00:35
Random Seed	342234091

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.643	5%	0.78
Maximum	2.198	10%	0.82
Mean	1.061	15%	0.84
Std Dev	0.240	20%	0.87
Variance	0.057	25%	0.89
Skewness	1.214	30%	0.91
Kurtosis	4.426	35%	0.93
Median	1.002	40%	0.95
Mode	0.812	45%	0.98
Left X	0.661	50%	1.00
Left P	0.01%	55%	1.03
Right X	2.17	60%	1.06
Right P	99.99%	65%	1.09
Diff X	1.51	70%	1.13
Diff P	99.98%	75%	1.18
#Errors	0	80%	1.23
Filter Min		85%	1.30
Filter Max		90%	1.40
#Filtered	0	95%	1.56

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.949	0.938
#2	Concrete area	0.236	0.248
#3	Backfill depth	0.200	0.188
#4	Total section area	-0.056	0.165
#5	Concrete unit weight	0.048	0.058
#6	Saturated ballast unit weight	0.000	0.001
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the BO Aqueduct Section - Bulk Unit Weights for Crown Backfill

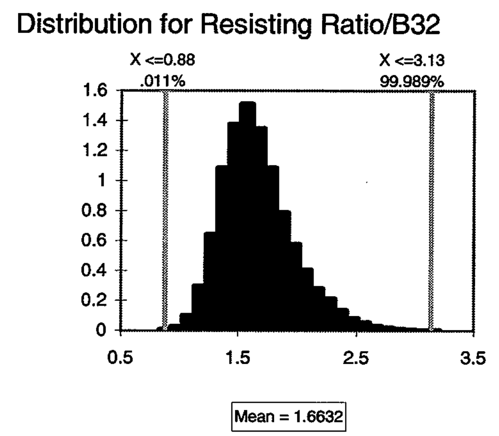
Summary of Output Statistics

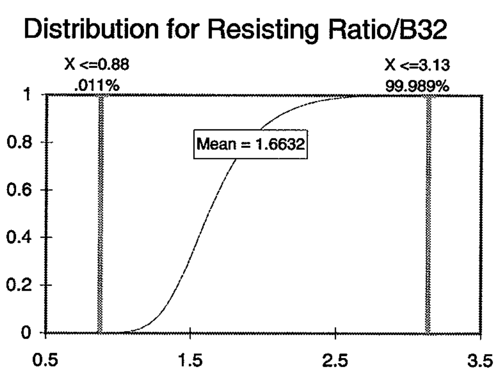
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	6.272	1.000	n/a	35.085	53.204	0.828	0.000	62.781	1.000
Maximum	127.880	34.850	n/a	56.043	207.561	3.210	0.208	207.561	3.210
Mean	51.317	10.737	n/a	45.379	107.432	1.663	0.000	107.439	1.663
Standard Deviation	14.952	6.790	n/a	3.753	19.730	0.303	0.003	19.716	0.303
Coeff. Of Variation	0.291	0.632	n/a	0.083	0.184	0.182	28.529	0.184	0.182
Variance	223.577	46.109	n/a	14.088	389.287	0.092	0.000	388.707	0.092
Skewness	0.525	1.195	n/a	0.031	0.772	0.784	35.903	0.778	0.790
Kurtosis	3.704	4.055	n/a	2.445	3.900	3.927	1488.343	3.895	3.922
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	29.897	5.601	n/a	46.939	81.864	1.133	0.000	81.864	1.000
5%	28.951	3.071	n/a	39.163	79.850	1.241	0.000	79.850	1.241
10%	33.309	3.831	n/a	40.356	84.828	1.317	0.000	84.828	1.317
15%	36.471	4.473	n/a	41.246	88.251	1.370	0.000	88.251	1.370
20%	38.965	5.072	n/a	42.002	91.083	1.413	0.000	91.083	1.413
25%	41.063	5.660	n/a	42.671	93.650	1.452	0.000	93.650	1.452
30%	43.054	6.252	n/a	43.281	96.060	1.488	0.000	96.060	1.488
35%	44.857	6.862	n/a	43.851	98.278	1.522	0.000	98.278	1.522
40%	46.680	7.499	n/a	44.386	100.400	1.554	0.000	100.400	1.554
45%	48.454	8.175	n/a	44.860	102.557	1.588	0.000	102.557	1.588
50%	50.245	8.903	n/a	45.361	104.769	1.622	0.000	104.769	1.622
55%	51.952	9.696	n/a	45.858	107.130	1.657	0.000	107.130	1.657
60%	53.785	10.575	n/a	46.349	109.518	1.695	0.000	109.518	1.695
65%	55.746	11.565	n/a	46.865	112.061	1.735	0.000	112.061	1.735
70%	57.840	12.703	n/a	47.412	115.014	1.780	0.000	115.014	1.780
75%	60.104	14.044	n/a	48.031	118.413	1.830	0.000	118.413	1.830
80%	62.821	15.679	n/a	48.728	122.312	1.891	0.000	122.312	1.891
85%	66.186	17.770	n/a	49.524	127.257	1.968	0.000	127.257	1.968
90%	70.782	20.641	n/a	50.454	133.952	2.073	0.000	133.952	2.073
95%	77.716	25.136	n/a	51.686	144.430	2.235	0.000	144.430	2.235
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	8.415	1.151	n/a	35.553	56.844	0.877	0.000	63.239	1.000
Target #1 (Perc%)	0.011%	0.011%	n/a	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	124.193	34.802	n/a	55.562	204.748	3.135	0.156	204.748	3.135
Target #2 (Perc%)	99.989%	99.989%	n/a	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.208		
Target #3 (Perc%)						0.190%	100.000%		

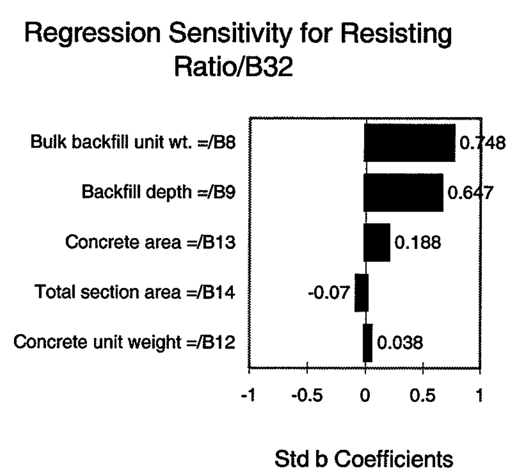
Monte Carlo Simulation Summary for the BO Aqueduct Section

(bulk unit weights for crown backfill)

Simulation Results for Resisting Ratio







Summary Information	
Workbook Name	analysis BO_bulk wts.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:21
Simulation Stop Time	4/29/2002 17:22
Simulation Duration	00:00:36
Random Seed	2081673222

Summary Statistics

Statistic	Value	%tile	Value
Minimum	0.828	5%	1.24
Maximum	3.210	10%	1.32
Mean	1.663	15%	1.37
Std Dev	0.303	20%	1.41
Variance	0.092	25%	1.45
Skewness	0.784	30%	1.49
Kurtosis	3.927	35%	1.52
Median	1.622	40%	1.55
Mode	1.133	45%	1.59
Left X	0.877	50%	1.62
Left P	0.01%	55%	1.66
Right X	3.13	60%	1.70
Right P	99.99%	65%	1.73
Diff X	2.26	70%	1.78
Diff P	99.98%	75%	1.83
#Errors	0	80%	1.89
Filter Min		85%	1.97
Filter Max		90%	2.07
#Filtered	0	95%	2.23

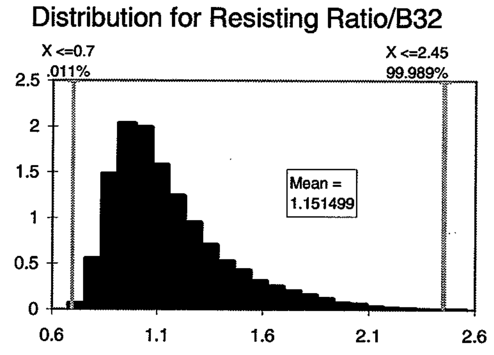
Sensitivity

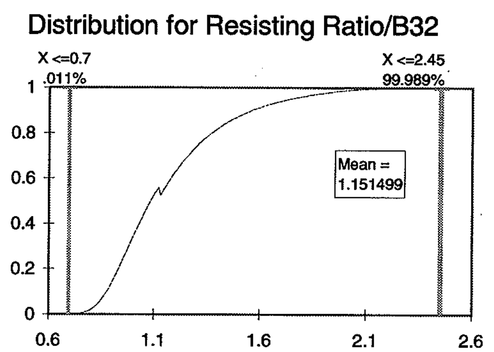
Rank	Name	Regr	Corr
#1	Bulk backfill unit wt.	0.748	0.682
#2	Backfill depth	0.647	0.657
#3	Concrete area	0.188	0.151
#4	Total section area	-0.070	0.083
#5	Concrete unit weight	0.038	0.047
#6	Bulk ballast unit weight	0.000	0.000
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

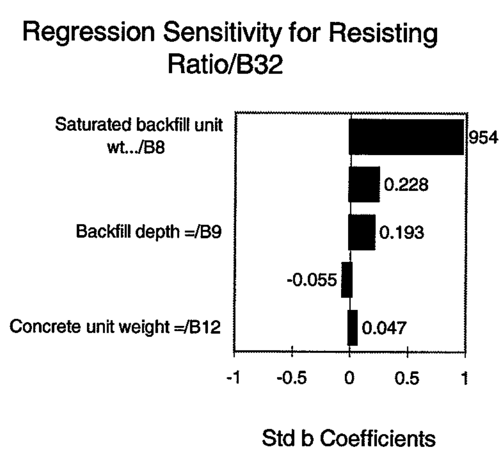
Monte Carlo Simulation Summary for the BP Aqueduct Section Summary of Output Statistics									
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.505	1.229	n/a	38.543	44.621	0.681	0.000	64.325	1.000
Maximum	75.594	41.845	n/a	61.374	171.892	2.559	0.643	171.892	2.559
Mean	13.818	12.893	n/a	49.730	76.441	1.151	0.061	78.464	1.182
Standard Deviation	9.603	8.154	n/a	4.103	17.820	0.267	0.111	15.927	0.238
Coeff. Of Variation	0.695	0.632	n/a	0.083	0.233	0.231	1.828	0.203	0.202
Variance	92.217	66.495	n/a	16.834	317.537	0.071	0.012	253.664	0.057
Skewness	1.496	1.195	n/a	0.029	1.185	1.206	1.923	1.624	1.643
Kurtosis	5.627	4.055	n/a	2.455	4.368	4.396	5.993	5.438	5.479
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	7.752	6.586	n/a	43.355	79.783	0.849	0.000	79.783	1.000
5%	3.442	3.688	n/a	42.918	55.273	0.839	0.000	65.255	1.000
10%	4.440	4.601	n/a	44.239	58.014	0.879	0.000	65.679	1.000
15%	5.289	5.371	n/a	45.234	60.109	0.908	0.000	66.021	1.000
20%	6.126	6.091	n/a	46.066	61.915	0.934	0.000	66.320	1.000
25%	6.878	6.797	n/a	46.800	63.570	0.959	0.000	66.616	1.000
30%	7.691	7.508	n/a	47.456	65.243	0.982	0.000	67.000	1.000
35%	8.473	8.240	n/a	48.071	66.827	4.055	0.000	67.506	1.006
40%	9.325	9.005	n/a	48.627	68.489	1.031	0.000	68.489	1.031
45%	10.221	9.817	n/a	49.157	70.277	1.058	0.000	70.277	1.058
50%	11.186	10.691	n/a	49.683	72.042	1.085	0.000	72.042	1.085
55%	12.254	11.644	n/a	50.203	74.184	1.116	0.000	74.184	1.116
60%	13.332	12.699	n/a	50.777	76.392	1.149	0.000	76.392	1.149
65%	14.661	13.888	n/a	51.362	78.862	1.186	0.000	78.862	1.186
70%	16.198	15.255	n/a	51.967	81.698	1.229	0.036	81.698	1.229
75%	18.017	16.865	n/a	52.642	85.048	1.280	0.082	85.048	1.280
80%	20.262	18.829	n/a	53.409	89.090	1.340	0.131	89.090	1.340
85%	23.102	21.338	n/a	54.268	94.580	1.422	0.183	94.580	1.422
90%	27.082	24.788	n/a	55.271	101.631	1.530	0.240	101.631	1.530
95%	33.478	30.185	n/a	56.631	112.959	1.704	0.319	112.959	1.704
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.757	1.376	0.000	38.904	45.388	0.695	0.000	64.348	1.000
Target #1 (Perc%)	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	68.490	41.798	0.000	61.181	163.124	2.452	0.607	163.124	2.452
Target #2 (Perc%)	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.643		
Target #3 (Perc%)						33.560%	100.000%		

Monte Carlo Simulation Summary for the BP Aqueduct Section

Simulation Results for Resisting Ratio







Summary Information	
Workbook Name	analysis BP section.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 17:33
Simulation Stop Time	4/29/2002 17:34
Simulation Duration	00:00:32
Random Seed	771846200

Summary Statistics

Statistic	Value	%tile	Value
Minimum	0.681	5%	0.84
Maximum	2.559	10%	0.88
Mean	1.151	15%	0.91
Std Dev	0.267	20%	0.93
Variance	0.071	25%	0.96
Skewness	1.206	30%	0.98
Kurtosis	4.055	35%	1.01
Median	1.085	40%	1.03
Mode	0.849	45%	1.06
Left X	0.695	50%	1.08
Left P	0.01%	55%	1.12
Right X	2.45	60%	1.15
Right P	99.99%	65%	1.19
Diff X	1.76	70%	1.23
Diff P	99.98%	75%	1.28
#Errors	0	80%	1.34
Filter Min		85%	1.42
Filter Max		90%	1.53
#Filtered	0	95%	1.70

Sensitivity

Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.954	0.942
#2	Concrete area	0.228	0.234
#3	Backfill depth	0.193	0.175
#4	Total section area	-0.055	0.151
#5	Concrete unit weight	0.047	0.058
#6	Saturated ballast unit weight	0.000	0.005
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

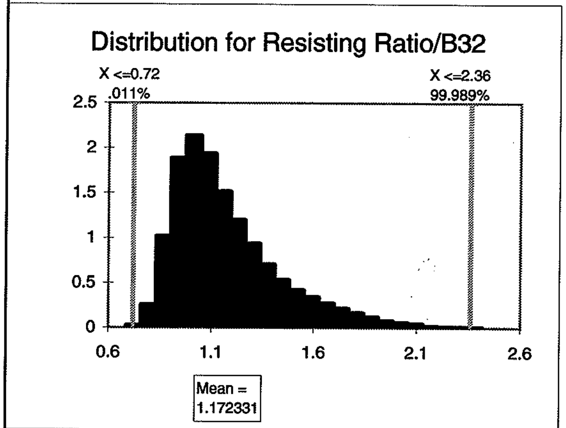
Monte Carlo Simulation Summary for the BPM Aqueduct Section

Summary of Output Statistics

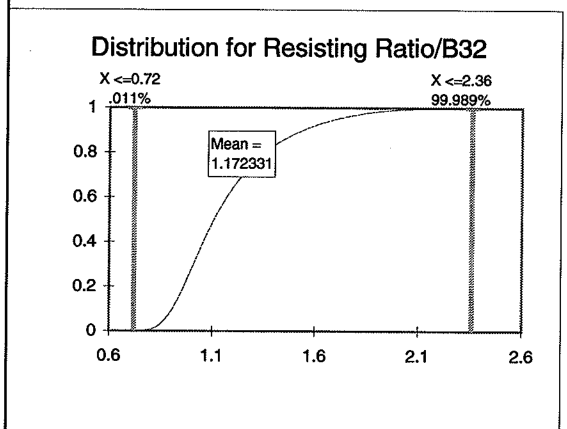
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.627	1.234	n/a	41.462	45.423	0.688	0.000	65.828	1.000
Maximum	71.909	40.237	n/a	66.902	166.568	2.418	0.614	166.568	2.418
Mean	13.817	12.397	n/a	53.428	79.642	1.172	0.044	81.099	1.194
Standard Deviation	9.613	7.841	n/a	4.420	17.587	0.257	0.091	16.141	0.236
Coeff. Of Variation	0.696	0.632	n/a	0.083	0.221	0.219	2.090	0.199	0.197
Variance	92.417	61.477	n/a	19.535	309.316	0.066	0.008	260.517	0.056
Skewness	1.487	1.195	n/a	0.036	1.171	1.196	2.321	1.522	1.544
Kurtosis	5.560	4.055	n/a	2.463	4.344	4.379	8.004	5.124	5.172
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	4.297	6.468	n/a	45.340	60.835	0.879	0.000	66.859	1.000
5%	3.475	3.546	n/a	46.118	58.632	0.870	0.000	66.836	1.000
10%	4.489	4.424	n/a	47.504	61.423	0.909	0.000	67.298	1.000
15%	5.318	5.165	n/a	48.564	63.469	0.938	0.000	67.677	1.000
20%	6.075	5.857	n/a	49.481	65.274	0.963	0.000	68.029	1.000
25%	6.841	6.535	n/a	50.286	66.965	0.986	0.000	68.451	1.000
30%	7.604	7.219	n/a	50.980	68.573	1.010	0.000	69.038	1.010
35%	8.428	7.923	n/a	51.643	70.191	1.033	0.000	70.191	1.033
40%	9.278	8.659	n/a	52.230	71.831	1.057	0.000	71.831	1.057
45%	10.148	9.440	n/a	52.822	73.585	1.082	0.000	73.585	1.082
50%	11.140	10.280	n/a	53.369	75.444	1.109	0.000	75.444	1.109
55%	12.168	11.196	n/a	53.970	77.377	1.138	0.000	77.377	1.138
60%	13.360	12.211	n/a	54.559	79.608	1.170	0.000	79.608	1.170
65%	14.680	13.354	n/a	55.182	82.096	1.206	0.000	82.096	1.206
70%	16.270	14.668	n/a	55.841	84.913	1.248	0.000	84.913	1.248
75%	18.129	16.216	n/a	56.555	88.161	1.296	0.028	88.161	1.296
80%	20.359	18.105	n/a	57.351	92.325	1.357	0.075	92.325	1.357
85%	23.162	20.519	n/a	58.271	97.469	1.434	0.127	97.469	1.434
90%	27.150	23.833	n/a	59.400	104.732	1.541	0.185	104.732	1.541
95%	33.630	29.020	n/a	60.875	115.611	1.701	0.264	115.611	1.701
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.883	1.318	n/a	42.009	47.542	0.720	0.000	65.848	1.000
Target #1 (Perc%)	0.011%	0.011%	n/a	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	66.631	40.191	n/a	65.833	162.487	2.359	0.574	162.487	2.359
Target #2 (Perc%)	99.989%	99.989%	n/a	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.614		
Target #3 (Perc%)						27.820%	100.000%		

Monte Carlo Simulation Summary for the BPM Aqueduct Section

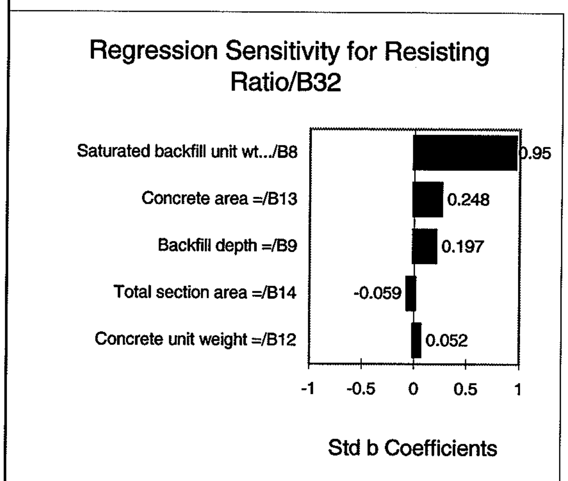
Simulation Results for Resisting Ratio



Summary Information	
Workbook Name	eliability analysis BPM .xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 18:26
Simulation Stop Time	4/29/2002 18:26
Simulation Duration	00:00:35
Random Seed	240946556



Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.688	5%	0.87
Maximum	2.418	10%	0.91
Mean	1.172	15%	0.94
Std Dev	0.257	20%	0.96
Variance	0.066	25%	0.99
Skewness	1.196	30%	1.01
Kurtosis	4.379	35%	1.03
Median	1.109	40%	1.06
Mode	0.879	45%	1.08
Left X	0.720	50%	1.11
Left P	0.01%	55%	1.14
Right X	2.36	60%	1.17
Right P	99.99%	65%	1.21
Diff X	1.64	70%	1.25
Diff P	99.98%	75%	1.30
#Errors	0	80%	1.36
Filter Min		85%	1.43
Filter Max		90%	1.54
#Filtered	0	95%	1.70



Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.950	0.934
#2	Concrete area	0.248	0.247
#3	Backfill depth	0.197	0.177
#4	Total section area	-0.059	0.161
#5	Concrete unit weight	0.052	0.065
#6	Saturated ballast unit weight	0.000	-0.001
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the RA Aqueduct Section

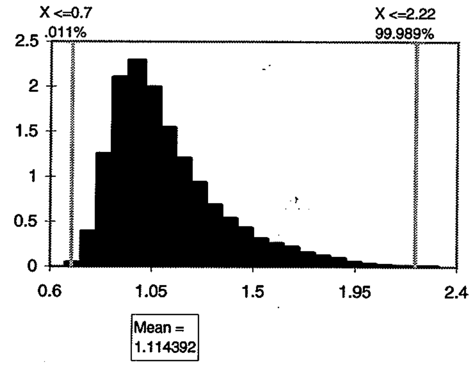
Summary of Output Statistics

Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.636	1.017	n/a	40.627	45.196	0.671	0.000	66.571	1.000
Maximum	66.108	39.109	n/a	64.713	161.838	2.318	0.751	161.838	2.318
Mean	12.555	12.052	n/a	51.948	76.555	1.114	0.080	78.976	1.150
Standard Deviation	8.759	7.622	n/a	4.283	16.607	0.239	0.133	14.424	0.208
Coeff. Of Variation	0.698	0.632	n/a	0.082	0.217	0.215	1.665	0.183	0.181
Variance	76.723	58.100	n/a	18.344	275.789	0.057	0.018	208.060	0.043
Skewness	1.482	1.195	n/a	0.026	1.170	1.193	1.693	1.707	1.727
Kurtosis	5.456	4.055	n/a	2.448	4.337	4.360	5.039	5.704	5.743
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	4.370	6.024	n/a	43.934	58.778	0.822	0.000	67.061	1.000
5%	3.150	3.448	n/a	44.839	56.592	0.831	0.000	67.474	1.000
10%	4.071	4.301	n/a	46.196	59.341	0.868	0.000	67.886	1.000
15%	4.850	5.021	n/a	47.242	61.401	0.896	0.000	68.215	1.000
20%	5.510	5.694	n/a	48.116	63.116	0.920	0.000	68.494	1.000
25%	6.191	6.353	n/a	48.871	64.683	0.942	0.000	68.756	1.000
30%	6.886	7.018	n/a	49.564	66.192	0.964	0.000	69.062	1.000
35%	7.660	7.702	n/a	50.198	67.709	0.985	0.000	69.422	1.000
40%	8.433	8.418	n/a	50.786	69.246	1.007	0.000	69.895	1.007
45%	9.228	9.177	n/a	51.370	70.805	1.030	0.000	70.805	1.030
50%	10.101	9.993	n/a	51.949	72.532	1.055	0.000	72.532	1.055
55%	11.053	10.884	n/a	52.484	74.461	1.082	0.000	74.461	1.082
60%	12.114	11.870	n/a	53.050	76.514	1.112	0.000	76.514	1.112
65%	13.338	12.981	n/a	53.660	78.886	1.147	0.034	78.886	1.147
70%	14.727	14.259	n/a	54.319	81.536	1.185	0.083	81.536	1.185
75%	16.368	15.765	n/a	55.006	84.568	1.229	0.132	84.568	1.229
80%	18.426	17.601	n/a	55.761	88.370	1.284	0.181	88.370	1.284
85%	21.083	19.947	n/a	56.666	93.388	1.357	0.235	93.388	1.357
90%	24.764	23.170	n/a	57.727	100.074	1.453	0.297	100.074	1.453
95%	30.791	28.212	n/a	59.080	110.708	1.611	0.380	110.708	1.611
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.800	1.276	n/a	41.094	46.765	0.695	0.000	66.597	1.000
Target #1 (Perc%)	0.011%	0.011%	n/a	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	61.128	39.068	n/a	63.807	152.483	2.217	0.724	152.483	2.217
Target #2 (Perc%)	99.989%	99.989%	n/a	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.751		
Target #3 (Perc%)						38.480%	100.000%		

Monte Carlo Simulation Summary for the RA Aqueduct Section

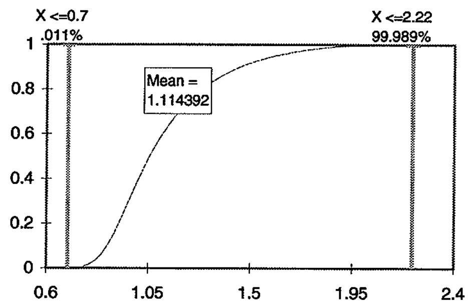
Simulation Results for Resisting Ratio

Distribution for Resisting Ratio/B32



Summary Information	
Workbook Name	reliability analysis RA.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 18:29
Simulation Stop Time	4/29/2002 18:30
Simulation Duration	00:00:35
Random Seed	507603147

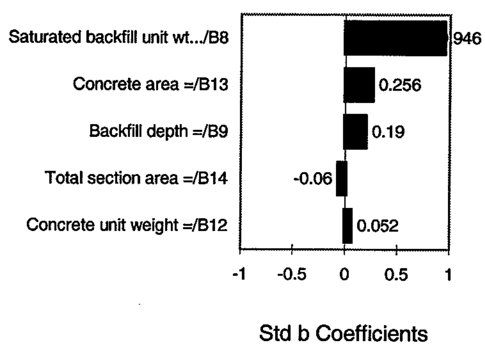
Distribution for Resisting Ratio/B32



Summary Statistics

Statistic	Value	%tile	Value
Minimum	0.671	5%	0.83
Maximum	2.318	10%	0.87
Mean	1.114	15%	0.90
Std Dev	0.239	20%	0.92
Variance	0.057	25%	0.94
Skewness	1.193	30%	0.96
Kurtosis	4.360	35%	0.98
Median	1.055	40%	1.01
Mode	0.822	45%	1.03
Left X	0.695	50%	1.05
Left P	0.01%	55%	1.08
Right X	2.22	60%	1.11
Right P	99.99%	65%	1.15
Diff X	1.52	70%	1.19
Diff P	99.98%	75%	1.23
#Errors	0	80%	1.28
Filter Min		85%	1.36
Filter Max		90%	1.45
#Filtered	0	95%	1.61

Regression Sensitivity for Resisting Ratio/B32



Sensitivity

Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.946	0.933
#2	Concrete area	0.256	0.266
#3	Backfill depth	0.190	0.179
#4	Total section area	-0.060	0.172
#5	Concrete unit weight	0.052	0.064
#6	Saturated ballast unit weight	0.000	0.000
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the RA Aqueduct Section - including shear resistance

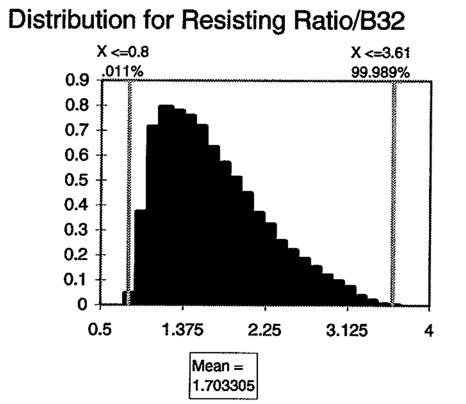
Summary of Output Statistics

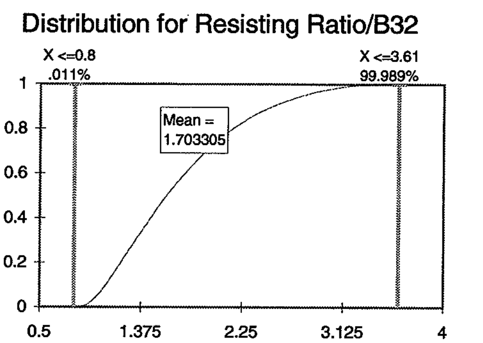
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.365	1.151	6.025	40.239	51.286	0.758	0.000	66.569	1.000
Maximum	70.253	39.108	95.972	64.368	255.176	3.683	0.539	255.176	3.683
Mean	12.579	12.052	40.408	51.948	116.987	1.703	0.007	117.194	1.706
Standard Deviation	8.790	7.622	22.721	4.290	37.805	0.549	0.037	37.519	0.545
Coeff. Of Variation	0.699	0.632	0.562	0.083	0.323	0.322	5.455	0.320	0.319
Variance	77.256	58.100	516.253	18.400	1429.211	0.302	0.001	1407.642	0.297
Skewness	1.480	1.195	0.486	0.026	0.742	0.744	6.713	0.774	0.776
Kurtosis	5.531	4.055	2.290	2.451	2.917	2.914	53.822	2.930	2.929
Number of Errors	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Mode	4.594	6.288	17.138	43.172	65.848	1.006	0.000	68.303	1.000
5%	3.168	3.448	9.579	44.839	68.474	0.999	0.000	69.250	1.000
10%	4.017	4.301	12.640	46.201	73.707	1.075	0.000	73.707	1.075
15%	4.778	5.021	15.633	47.255	78.287	1.139	0.000	78.287	1.139
20%	5.492	5.694	18.523	48.096	82.519	1.202	0.000	82.519	1.202
25%	6.207	6.353	21.457	48.871	86.875	1.264	0.000	86.875	1.264
30%	6.919	7.018	24.357	49.575	91.245	1.329	0.000	91.245	1.329
35%	7.624	7.702	27.366	50.201	95.781	1.394	0.000	95.781	1.394
40%	8.433	8.418	30.413	50.795	100.323	1.459	0.000	100.323	1.459
45%	9.278	9.177	33.673	51.383	104.754	1.525	0.000	104.754	1.525
50%	10.140	9.993	36.973	51.921	109.651	1.597	0.000	109.651	1.597
55%	11.043	10.884	40.381	52.469	115.122	1.674	0.000	115.122	1.674
60%	12.123	11.871	44.097	53.053	120.665	1.756	0.000	120.665	1.756
65%	13.404	12.981	47.951	53.657	126.593	1.846	0.000	126.593	1.846
70%	14.781	14.259	52.121	54.304	133.106	1.936	0.000	133.106	1.936
75%	16.454	15.765	56.586	55.016	140.574	2.045	0.000	140.574	2.045
80%	18.525	17.601	61.772	55.782	148.953	2.170	0.000	148.953	2.170
85%	21.161	19.946	67.363	56.664	159.342	2.316	0.000	159.342	2.316
90%	24.933	23.169	74.388	57.720	172.531	2.510	0.000	172.531	2.510
95%	30.874	28.214	82.779	59.136	191.351	2.786	0.003	191.351	2.786
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.738	1.283	6.032	40.834	53.464	0.796	0.000	66.640	1.000
Target #1 (Perc%)	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	63.722	39.070	95.908	63.576	247.249	3.609	0.476	247.249	3.609
Target #2 (Perc%)	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.539		
Target #3 (Perc%)						5.037%	100.000%		

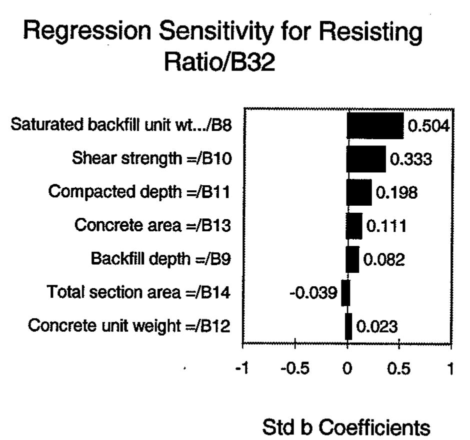
Monte Carlo Simulation Summary for the RA Aqueduct Section

(includes shear resistance)

Simulation Results for Resisting Ratio







Summary Information	
Workbook Name	lity analysis RA_shear.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	8
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 18:55
Simulation Stop Time	4/29/2002 18:56
Simulation Duration	00:00:55
Random Seed	211740647

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.758	5%	1.00
Maximum	3.683	10%	1.07
Mean	1.703	15%	1.14
Std Dev	0.549	20%	1.20
Variance	0.302	25%	1.26
Skewness	0.744	30%	1.33
Kurtosis	2.914	35%	1.39
Median	1.597	40%	1.46
Mode	1.006	45%	1.53
Left X	0.796	50%	1.60
Left P	0.01%	55%	1.67
Right X	3.61	60%	1.76
Right P	99.99%	65%	1.85
Diff X	2.81	70%	1.94
Diff P	99.98%	75%	2.04
#Errors	0	80%	2.17
Filter Min		85%	2.32
Filter Max		90%	2.51
#Filtered	0	95%	2.79

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.504	0.956
#2	Shear strength	0.333	0.955
#3	Compacted depth	0.198	0.950
#4	Concrete area	0.111	0.092
#5	Backfill depth	0.082	0.082
#6	Total section area	-0.039	0.051
#7	Concrete unit weight	0.023	0.027
#8	Saturated ballast unit weight	0.000	0.002
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

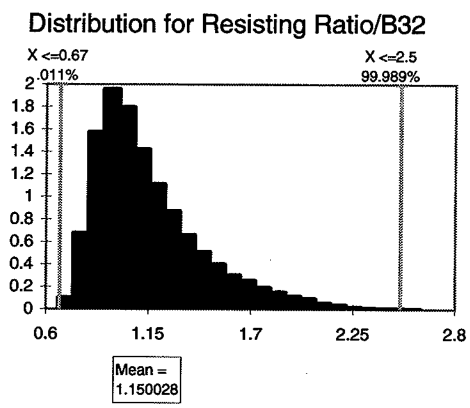
Monte Carlo Simulation Summary for the RE Aqueduct Section

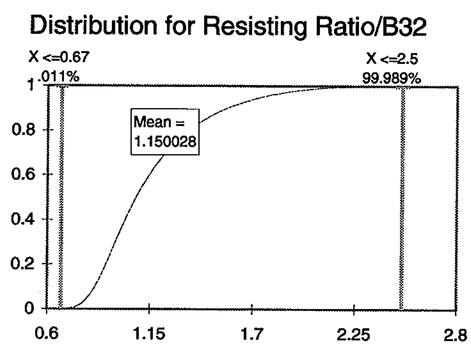
Summary of Output Statistics

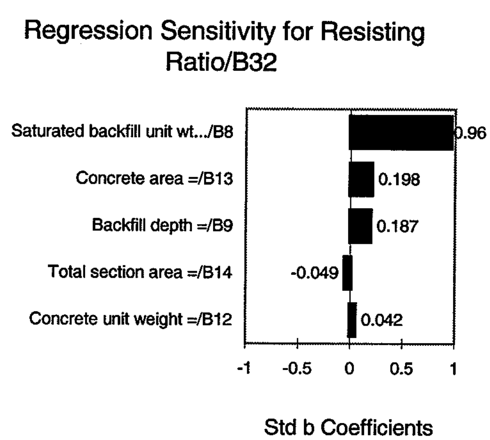
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.581	1.442	n/a	37.096	43.468	0.667	0.000	64.838	1.000
Maximum	78.682	47.572	n/a	59.416	175.616	2.616	0.647	175.616	2.616
Mean	14.558	14.658	n/a	47.737	76.953	1.150	0.071	79.446	1.187
Standard Deviation	10.164	9.270	n/a	3.940	19.400	0.288	0.122	17.156	0.255
Coeff. Of Variation	0.698	0.632	n/a	0.083	0.252	0.251	1.710	0.216	0.215
Variance	103.301	85.938	n/a	15.520	376.353	0.083	0.015	294.320	0.065
Skewness	1.511	1.195	n/a	0.027	1.216	1.231	1.729	1.685	1.699
Kurtosis	5.671	4.055	n/a	2.459	4.437	4.451	5.124	5.654	5.678
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	6.843	7.487	n/a	40.029	53.060	0.826	0.000	65.629	1.000
5%	3.672	4.193	n/a	41.181	54.258	0.816	0.000	65.771	1.000
10%	4.698	5.230	n/a	42.437	57.085	0.857	0.000	66.180	1.000
15%	5.604	6.106	n/a	43.399	59.312	0.889	0.000	66.510	1.000
20%	6.459	6.925	n/a	44.216	61.163	0.915	0.000	66.799	1.000
25%	7.244	7.727	n/a	44.914	62.949	0.941	0.000	67.079	1.000
30%	8.049	8.535	n/a	45.555	64.629	0.966	0.000	67.407	1.000
35%	8.873	9.367	n/a	46.157	66.392	0.991	0.000	67.817	1.000
40%	9.758	10.237	n/a	46.698	68.176	1.018	0.000	68.448	1.018
45%	10.695	11.160	n/a	47.218	70.013	1.045	0.000	70.013	1.045
50%	11.698	12.154	n/a	47.727	71.963	1.076	0.000	71.963	1.076
55%	12.817	13.237	n/a	48.235	74.174	1.108	0.000	74.174	1.108
60%	14.044	14.437	n/a	48.751	76.727	1.145	0.000	76.727	1.145
65%	15.447	15.788	n/a	49.301	79.401	1.186	0.016	79.401	1.186
70%	17.076	17.342	n/a	49.885	82.601	1.234	0.065	82.601	1.234
75%	19.054	19.173	n/a	50.532	86.435	1.289	0.112	86.435	1.289
80%	21.315	21.406	n/a	51.248	90.784	1.355	0.162	90.784	1.355
85%	24.353	24.259	n/a	52.047	96.738	1.443	0.211	96.738	1.443
90%	28.614	28.179	n/a	53.029	104.536	1.560	0.270	104.536	1.560
95%	35.402	34.311	n/a	54.327	116.923	1.745	0.349	116.923	1.745
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.877	1.570	n/a	37.635	43.830	0.672	0.000	64.867	1.000
Target #1 (Perc%)	0.011%	0.011%	n/a	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	75.534	47.521	n/a	58.778	169.720	2.504	0.628	169.720	2.504
Target #2 (Perc%)	99.989%	99.989%	n/a	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.647		
Target #3 (Perc%)						36.587%	100.000%		

Monte Carlo Simulation Summary for the RE Aqueduct Section

Simulation Results for Resisting Ratio







Summary Information	
Workbook Name	reliability analysis RE.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 18:32
Simulation Stop Time	4/29/2002 18:33
Simulation Duration	00:00:33
Random Seed	684511398

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.667	5%	0.82
Maximum	2.616	10%	0.86
Mean	1.150	15%	0.89
Std Dev	0.288	20%	0.91
Variance	0.083	25%	0.94
Skewness	1.231	30%	0.97
Kurtosis	4.451	35%	0.99
Median	1.076	40%	1.02
Mode	0.826	45%	1.05
Left X	0.672	50%	1.08
Left P	0.01%	55%	1.11
Right X	2.50	60%	1.14
Right P	99.99%	65%	1.19
Diff X	1.83	70%	1.23
Diff P	99.98%	75%	1.29
#Errors	0	80%	1.36
Filter Min		85%	1.44
Filter Max		90%	1.56
#Filtered	0	95%	1.75

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.960	0.952
#2	Concrete area	0.198	0.210
#3	Backfill depth	0.187	0.172
#4	Total section area	-0.049	0.137
#5	Concrete unit weight	0.042	0.044
#6	Saturated ballast unit weight	0.000	0.003
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

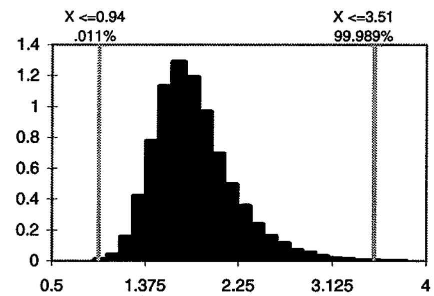
Monte Carlo Simulation Summary for the RE Aqueduct Section - bulk unit weights for crown backfill Summary of Output Statistics									
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	7.827	1.400	n/a	37.116	58.966	0.899	0.000	61.142	0.932
Maximum	160.821	47.576	n/a	59.064	258.875	3.818	0.106	258.875	3.818
Mean	60.074	14.658	n/a	47.737	122.469	1.831	0.000	122.470	1.831
Standard Deviation	17.640	9.270	n/a	3.942	24.166	0.360	0.001	24.165	0.360
Coeff. Of Variation	0.294	0.632	n/a	0.083	0.197	0.196	51.618	0.197	0.196
Variance	311.178	85.938	n/a	15.541	584.008	0.129	0.000	583.954	0.129
Skewness	0.550	1.195	n/a	0.024	0.836	0.850	64.145	0.837	0.851
Kurtosis	3.748	4.055	n/a	2.445	4.031	4.062	4566.671	4.031	4.061
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	31.040	7.647	n/a	41.027	87.872	1.313	0.000	87.872	1.313
5%	33.721	4.193	n/a	41.191	89.049	1.334	0.000	89.049	1.334
10%	39.033	5.231	n/a	42.462	95.098	1.426	0.000	95.098	1.426
15%	42.602	6.106	n/a	43.392	99.239	1.486	0.000	99.239	1.486
20%	45.427	6.925	n/a	44.185	102.691	1.537	0.000	102.691	1.537
25%	48.026	7.727	n/a	44.912	105.729	1.581	0.000	105.729	1.581
30%	50.309	8.535	n/a	45.546	108.443	1.623	0.000	108.443	1.623
35%	52.493	9.367	n/a	46.135	111.129	1.662	0.000	111.129	1.662
40%	54.623	10.237	n/a	46.690	113.716	1.700	0.000	113.716	1.700
45%	56.628	11.160	n/a	47.217	116.334	1.739	0.000	116.334	1.739
50%	58.649	12.154	n/a	47.723	119.070	1.778	0.000	119.070	1.778
55%	60.715	13.237	n/a	48.234	121.866	1.820	0.000	121.866	1.820
60%	62.922	14.437	n/a	48.755	124.795	1.863	0.000	124.795	1.863
65%	65.129	15.788	n/a	49.306	127.889	1.910	0.000	127.889	1.910
70%	67.597	17.342	n/a	49.916	131.348	1.962	0.000	131.348	1.962
75%	70.546	19.174	n/a	50.552	135.602	2.025	0.000	135.602	2.025
80%	73.728	21.407	n/a	51.267	140.411	2.096	0.000	140.411	2.096
85%	77.720	24.259	n/a	52.039	146.400	2.186	0.000	146.400	2.186
90%	82.761	28.178	n/a	53.051	154.770	2.310	0.000	154.770	2.310
95%	91.302	34.314	n/a	54.349	168.137	2.516	0.000	168.137	2.516
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	9.925	1.554	n/a	37.462	62.862	0.937	0.000	64.085	0.957
Target #1 (Perc%)	0.011%	0.011%	n/a	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	144.929	47.519	n/a	58.319	238.538	3.512	0.070	238.538	3.512
Target #2 (Perc%)	99.989%	99.989%	n/a	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.106		
Target #3 (Perc%)						0.067%	100.000%		

Monte Carlo Simulation Summary for the RE Aqueduct Section

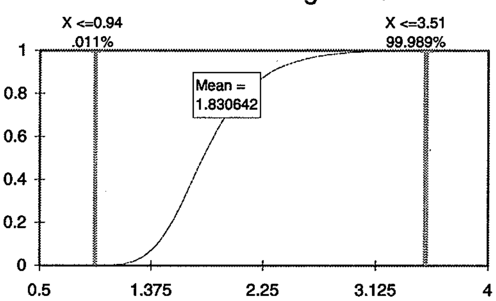
(bulk unit weights for crown backfill)

Simulation Results for Resisting Ratio

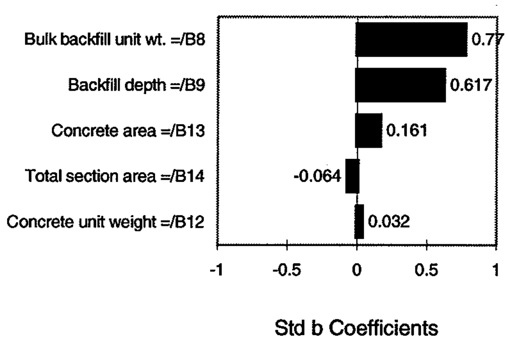
Distribution for Resisting Ratio/B32



Distribution for Resisting Ratio/B32



Regression Sensitivity for Resisting Ratio/B32



Summary Information	
Workbook Name	ability analysis RE_bulk.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	5/12/2002 13:59
Simulation Stop Time	5/12/2002 13:59
Simulation Duration	00:00:31
Random Seed	1059481749

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.899	5%	1.33
Maximum	3.818	10%	1.43
Mean	1.831	15%	1.49
Std Dev	0.360	20%	1.54
Variance	0.129	25%	1.58
Skewness	0.850	30%	1.62
Kurtosis	4.062	35%	1.66
Median	1.778	40%	1.70
Mode	1.313	45%	1.74
Left X	0.937	50%	1.78
Left P	0.01%	55%	1.82
Right X	3.51	60%	1.86
Right P	99.99%	65%	1.91
Diff X	2.57	70%	1.96
Diff P	99.98%	75%	2.02
#Errors	0	80%	2.10
Filter Min		85%	2.19
Filter Max		90%	2.31
#Filtered	0	95%	2.52

Sensitivity			
Rank	Name	Regr	Corr
#1	Bulk backfill unit wt.	0.770	0.713
#2	Backfill depth	0.617	0.640
#3	Concrete area	0.161	0.121
#4	Total section area	-0.064	0.061
#5	Concrete unit weight	0.032	0.040
#6	Bulk ballast unit weight	0.000	0.003
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the RE Aqueduct Section - includes shear resistance

Summary of Output Statistics

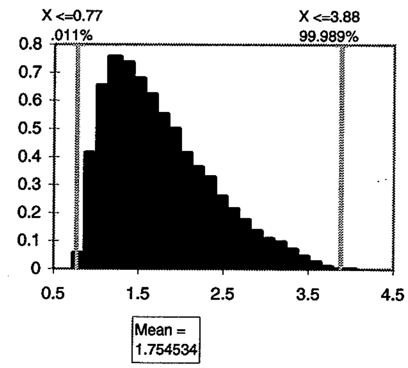
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.599	1.402	6.007	36.660	47.764	0.734	0.000	64.897	1.000
Maximum	78.574	47.577	95.964	59.698	271.966	4.059	0.508	271.966	4.059
Mean	14.575	14.658	40.410	47.738	117.380	1.755	0.008	117.648	1.759
Standard Deviation	10.163	9.270	22.731	3.944	40.573	0.606	0.038	40.212	0.600
Coeff. Of Variation	0.697	0.632	0.563	0.083	0.346	0.345	4.997	0.342	0.341
Variance	103.292	85.938	516.684	15.553	1646.163	0.367	0.001	1617.041	0.360
Skewness	1.492	1.195	0.487	0.024	0.781	0.783	6.078	0.819	0.821
Kurtosis	5.571	4.055	2.291	2.454	3.015	3.015	44.318	3.036	3.037
Number of Errors	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Mode	3.842	7.487	17.957	41.312	69.986	1.035	0.000	68.462	1.000
5%	3.615	4.193	9.551	41.163	65.693	0.983	0.000	67.247	1.000
10%	4.678	5.230	12.716	42.427	71.288	1.067	0.000	71.288	1.067
15%	5.594	6.106	15.510	43.429	76.328	1.142	0.000	76.328	1.142
20%	6.409	6.925	18.553	44.205	80.813	1.207	0.000	80.813	1.207
25%	7.214	7.727	21.420	44.902	85.074	1.273	0.000	85.074	1.273
30%	8.047	8.536	24.398	45.549	89.753	1.341	0.000	89.753	1.341
35%	8.899	9.367	27.342	46.145	94.301	1.409	0.000	94.301	1.409
40%	9.814	10.237	30.456	46.701	99.194	1.482	0.000	99.194	1.482
45%	10.761	11.160	33.683	47.216	104.081	1.557	0.000	104.081	1.557
50%	11.774	12.154	36.985	47.716	109.477	1.637	0.000	109.477	1.637
55%	12.888	13.237	40.487	48.206	114.986	1.717	0.000	114.986	1.717
60%	14.090	14.437	44.084	48.723	120.989	1.807	0.000	120.989	1.807
65%	15.496	15.788	47.922	49.289	127.171	1.901	0.000	127.171	1.901
70%	17.117	17.342	52.104	49.916	134.294	2.007	0.000	134.294	2.007
75%	18.991	19.174	56.663	50.571	142.440	2.128	0.000	142.440	2.128
80%	21.421	21.406	61.807	51.251	151.671	2.268	0.000	151.671	2.268
85%	24.398	24.259	67.451	52.082	162.397	2.428	0.000	162.397	2.428
90%	28.578	28.181	74.469	53.075	176.520	2.636	0.000	176.520	2.636
95%	35.576	34.313	82.937	54.321	197.289	2.948	0.031	197.289	2.948
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.708	1.590	6.013	37.537	50.653	0.771	0.000	64.957	1.000
Target #1 (Perc%)	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	71.597	47.524	95.945	58.637	258.905	3.884	0.459	258.905	3.884
Target #2 (Perc%)	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.508		
Target #3 (Perc%)						5.823%	100.000%		

Monte Carlo Simulation Summary for the RE Aqueduct Section

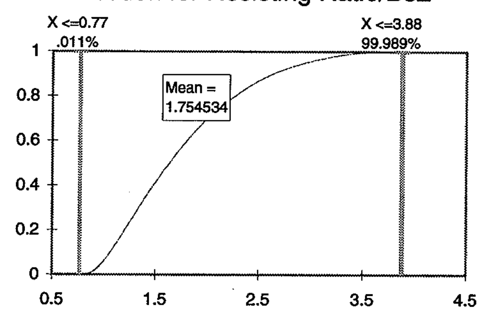
(includes shear resistance)

Simulation Results for Resisting Ratio

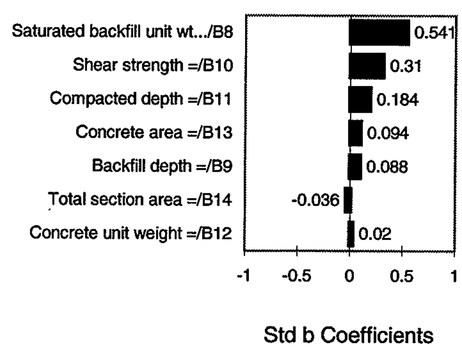
Distribution for Resisting Ratio/B32



Distribution for Resisting Ratio/B32



Regression Sensitivity for Resisting Ratio/B32



Summary Information	
Workbook Name	lity analysis RE_shear.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	8
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 19:00
Simulation Stop Time	4/29/2002 19:01
Simulation Duration	00:00:42
Random Seed	552867476

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.734	5%	0.98
Maximum	4.059	10%	1.07
Mean	1.755	15%	1.14
Std Dev	0.606	20%	1.21
Variance	0.367	25%	1.27
Skewness	0.783	30%	1.34
Kurtosis	3.015	35%	1.41
Median	1.637	40%	1.48
Mode	1.035	45%	1.56
Left X	0.771	50%	1.64
Left P	0.01%	55%	1.72
Right X	3.88	60%	1.81
Right P	99.99%	65%	1.90
Diff X	3.11	70%	2.01
Diff P	99.98%	75%	2.13
#Errors	0	80%	2.27
Filter Min		85%	2.43
Filter Max		90%	2.64
#Filtered	0	95%	2.95

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.541	0.962
#2	Shear strength	0.310	0.954
#3	Compacted depth	0.184	0.949
#4	Concrete area	0.094	0.073
#5	Backfill depth	0.088	0.080
#6	Total section area	-0.036	0.041
#7	Concrete unit weight	0.020	0.023
#8	Saturated ballast unit weight	0.000	-0.006
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the RG Aqueduct Section

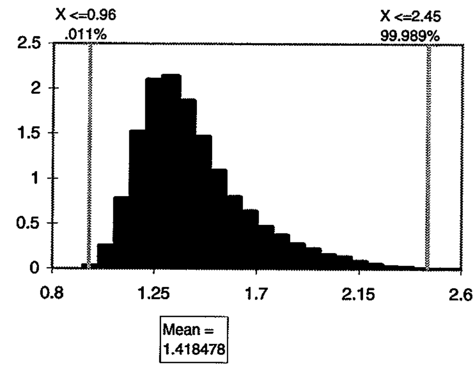
Summary of Output Statistics

Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.641	1.257	n/a	72.189	79.113	0.939	0.000	83.062	1.000
Maximum	77.587	47.901	n/a	113.083	226.895	2.598	0.135	226.895	2.598
Mean	14.523	14.761	n/a	92.019	121.302	1.418	0.000	121.304	1.418
Standard Deviation	10.143	9.336	n/a	7.589	20.511	0.236	0.002	20.508	0.236
Coeff. Of Variation	0.698	0.632	n/a	0.082	0.169	0.166	34.831	0.169	0.166
Variance	102.877	87.157	n/a	57.586	420.717	0.055	0.000	420.578	0.055
Skewness	1.496	1.195	n/a	0.024	1.029	1.073	45.352	1.031	1.075
Kurtosis	5.556	4.055	n/a	2.441	4.081	4.140	2402.603	4.081	4.140
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	4.814	7.540	n/a	75.735	90.658	1.078	0.000	90.658	1.000
5%	3.653	4.222	n/a	79.446	95.184	1.124	0.000	95.184	1.124
10%	4.682	5.267	n/a	81.811	99.095	1.167	0.000	99.095	1.167
15%	5.541	6.150	n/a	83.680	102.015	1.199	0.000	102.015	1.199
20%	6.367	6.974	n/a	85.206	104.578	1.227	0.000	104.578	1.227
25%	7.216	7.782	n/a	86.556	106.753	1.251	0.000	106.753	1.251
30%	8.033	8.596	n/a	87.785	108.861	1.274	0.000	108.861	1.274
35%	8.854	9.433	n/a	88.956	110.906	1.296	0.000	110.906	1.296
40%	9.737	10.310	n/a	90.025	112.948	1.320	0.000	112.948	1.320
45%	10.666	11.240	n/a	91.004	115.117	1.344	0.000	115.117	1.344
50%	11.662	12.240	n/a	91.993	117.285	1.368	0.000	117.285	1.368
55%	12.810	13.330	n/a	92.958	119.531	1.394	0.000	119.531	1.394
60%	14.055	14.539	n/a	93.967	121.901	1.423	0.000	121.901	1.423
65%	15.409	15.900	n/a	95.036	124.605	1.454	0.000	124.605	1.454
70%	17.030	17.464	n/a	96.172	127.899	1.491	0.000	127.899	1.491
75%	18.899	19.309	n/a	97.426	131.601	1.535	0.000	131.601	1.535
80%	21.258	21.558	n/a	98.865	136.226	1.589	0.000	136.226	1.589
85%	24.287	24.431	n/a	100.418	142.166	1.658	0.000	142.166	1.658
90%	28.613	28.377	n/a	102.261	150.059	1.752	0.000	150.059	1.752
95%	35.481	34.555	n/a	104.650	162.562	1.898	0.000	162.562	1.898
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.861	1.580	n/a	72.394	80.871	0.962	0.000	83.177	1.000
Target #1 (Perc%)	0.011%	0.011%	n/a	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	73.545	47.861	n/a	112.638	211.173	2.452	0.092	211.173	2.452
Target #2 (Perc%)	99.989%	99.989%	n/a	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.135		
Target #3 (Perc%)						0.143%	100.000%		

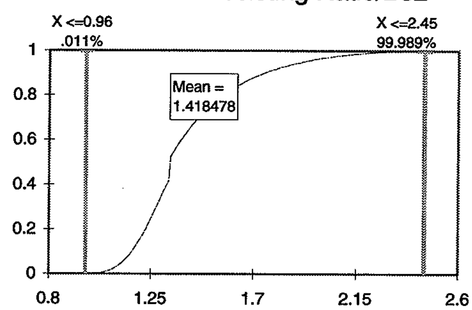
Monte Carlo Simulation Summary for the RG Aqueduct Section

Simulation Results for Resisting Ratio

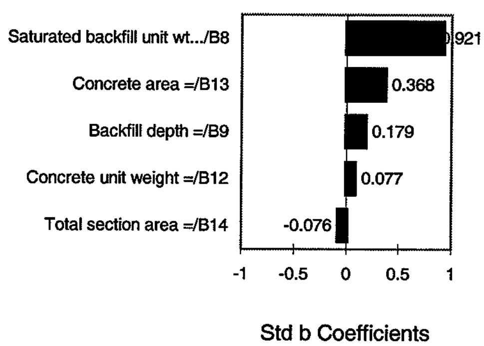
Distribution for Resisting Ratio/B32



Distribution for Resisting Ratio/B32



Regression Sensitivity for Resisting Ratio/B32



Summary Information	
Workbook Name	reliability analysis RG.xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 18:35
Simulation Stop Time	4/29/2002 18:35
Simulation Duration	00:00:33
Random Seed	871003896

Summary Statistics

Statistic	Value	%tile	Value
Minimum	0.939	5%	1.12
Maximum	2.598	10%	1.17
Mean	1.418	15%	1.20
Std Dev	0.236	20%	1.23
Variance	0.055	25%	1.25
Skewness	1.073	30%	1.27
Kurtosis	4.140	35%	1.30
Median	1.368	40%	1.32
Mode	1.078	45%	1.34
Left X	0.962	50%	1.37
Left P	0.01%	55%	1.39
Right X	2.45	60%	1.42
Right P	99.99%	65%	1.45
Diff X	1.49	70%	1.49
Diff P	99.98%	75%	1.53
#Errors	0	80%	1.59
Filter Min		85%	1.66
Filter Max		90%	1.75
#Filtered	0	95%	1.90

Sensitivity

Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.921	0.889
#2	Concrete area	0.368	0.369
#3	Backfill depth	0.179	0.160
#4	Concrete unit weight	0.077	0.080
#5	Total section area	-0.076	0.249
#6	Saturated ballast unit weight	0.000	0.005
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

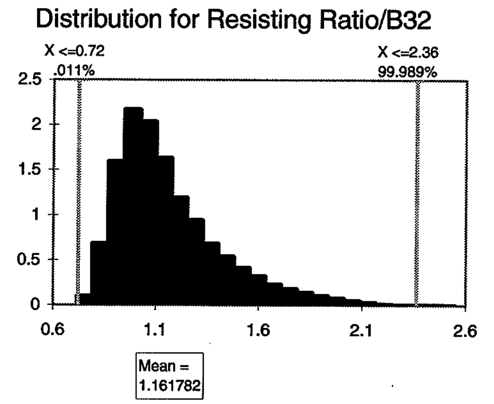
Monte Carlo Simulation Summary for the RPM Aqueduct Section

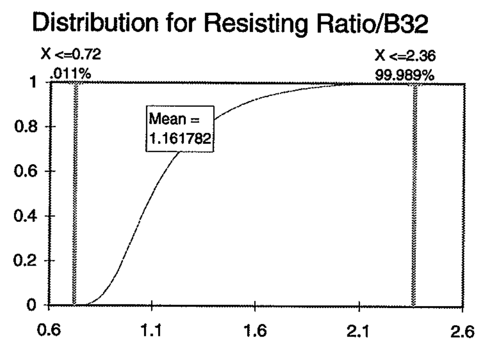
Summary of Output Statistics

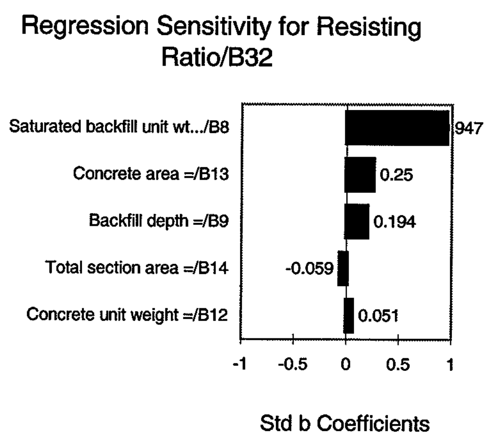
Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.654	1.069	n/a	42.051	48.678	0.719	0.000	67.594	1.000
Maximum	78.033	40.925	n/a	68.031	180.007	2.547	0.601	180.007	2.547
Mean	13.953	12.611	n/a	54.478	81.041	1.162	0.049	82.685	1.185
Standard Deviation	9.762	7.976	n/a	4.496	17.919	0.255	0.099	16.319	0.232
Coeff. Of Variation	0.700	0.632	n/a	0.083	0.221	0.219	2.011	0.197	0.196
Variance	95.296	63.613	n/a	20.212	321.105	0.065	0.010	266.295	0.054
Skewness	1.503	1.195	n/a	0.037	1.195	1.219	2.205	1.579	1.600
Kurtosis	5.636	4.055	n/a	2.448	4.444	4.476	7.322	5.338	5.384
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	4.183	6.304	n/a	45.756	63.669	0.867	0.000	68.563	1.000
5%	3.487	3.608	n/a	47.034	59.627	0.862	0.000	68.601	1.000
10%	4.497	4.500	n/a	48.484	62.512	0.901	0.000	69.067	1.000
15%	5.322	5.254	n/a	49.529	64.756	0.931	0.000	69.438	1.000
20%	6.122	5.958	n/a	50.461	66.539	0.956	0.000	69.782	1.000
25%	6.898	6.648	n/a	51.255	68.218	0.978	0.000	70.165	1.000
30%	7.687	7.343	n/a	51.982	69.894	1.002	0.000	70.683	1.002
35%	8.504	8.059	n/a	52.632	71.483	1.025	0.000	71.529	1.025
40%	9.344	8.808	n/a	53.250	73.151	1.047	0.000	73.151	1.047
45%	10.215	9.602	n/a	53.851	74.843	1.072	0.000	74.843	1.072
50%	11.201	10.457	n/a	54.446	76.718	1.098	0.000	76.718	1.098
55%	12.280	11.389	n/a	54.997	78.662	1.126	0.000	78.662	1.126
60%	13.449	12.421	n/a	55.600	80.874	1.158	0.000	80.874	1.158
65%	14.735	13.584	n/a	56.253	83.436	1.194	0.000	83.436	1.194
70%	16.365	14.920	n/a	56.947	86.335	1.236	0.000	86.335	1.236
75%	18.264	16.496	n/a	57.685	89.770	1.284	0.045	89.770	1.284
80%	20.608	18.417	n/a	58.492	93.829	1.343	0.091	93.829	1.343
85%	23.509	20.872	n/a	59.437	99.166	1.419	0.142	99.166	1.419
90%	27.443	24.244	n/a	60.592	106.544	1.522	0.205	106.544	1.522
95%	33.914	29.521	n/a	62.049	117.661	1.686	0.286	117.661	1.686
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.955	1.346	0.000	42.610	49.721	0.723	0.000	67.616	1.000
Target #1 (Perc%)	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	68.561	40.884	0.000	66.805	166.442	2.363	0.584	166.442	2.363
Target #2 (Perc%)	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.601		
Target #3 (Perc%)						29.600%	100.000%		

Monte Carlo Simulation Summary for the RPM Aqueduct Section

Simulation Results for Resisting Ratio







Summary Information	
Workbook Name	liability analysis RPM .xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 18:38
Simulation Stop Time	4/29/2002 18:39
Simulation Duration	00:00:35
Random Seed	1144426023

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.719	5%	0.86
Maximum	2.547	10%	0.90
Mean	1.162	15%	0.93
Std Dev	0.255	20%	0.96
Variance	0.065	25%	0.98
Skewness	1.219	30%	1.00
Kurtosis	4.476	35%	1.02
Median	1.098	40%	1.05
Mode	0.867	45%	1.07
Left X	0.723	50%	1.10
Left P	0.01%	55%	1.13
Right X	2.36	60%	1.16
Right P	99.99%	65%	1.19
Diff X	1.64	70%	1.24
Diff P	99.98%	75%	1.28
#Errors	0	80%	1.34
Filter Min		85%	1.42
Filter Max		90%	1.52
#Filtered	0	95%	1.69

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.947	0.935
#2	Concrete area	0.250	0.254
#3	Backfill depth	0.194	0.184
#4	Total section area	-0.059	0.171
#5	Concrete unit weight	0.051	0.048
#6	Saturated ballast unit weight	0.000	0.014
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			

Monte Carlo Simulation Summary for the RRM Aqueduct Section

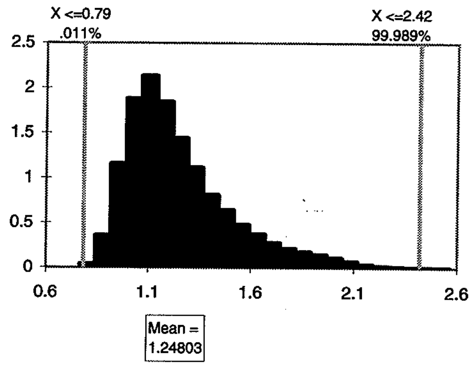
Summary of Output Statistics

Statistics	Crown Backfill Weight (kN/m)	Arch Leg Backfill Weight (kN/m)	Shear Resistance (kN/m)	Aqueduct Weight (kN/m)	Total Resisting Force (kN/m)	Resisting Ratio	Minimum Granular Ballast Depth (m)	New Total Resisting Force (kN/m)	New Resisting Ratio
Minimum	0.662	1.381	n/a	48.794	54.863	0.767	0.000	71.182	1.000
Maximum	77.097	46.760	n/a	78.425	189.625	2.568	0.535	189.625	2.568
Mean	13.949	14.410	n/a	63.328	91.686	1.248	0.015	92.196	1.255
Standard Deviation	9.775	9.113	n/a	5.221	19.123	0.258	0.051	18.519	0.250
Coeff. Of Variation	0.701	0.632	n/a	0.082	0.209	0.207	3.380	0.201	0.199
Variance	95.547	83.055	n/a	27.257	365.705	0.066	0.003	342.954	0.062
Skewness	1.512	1.195	n/a	0.021	1.163	1.189	4.175	1.318	1.342
Kurtosis	5.627	4.055	n/a	2.445	4.346	4.375	22.345	4.596	4.632
Number of Errors	0.000	0.000	n/a	0.000	0.000	0.000	0.000	0.000	0.000
Mode	3.859	7.203	n/a	53.226	67.436	0.908	0.000	72.598	1.000
5%	3.507	4.122	n/a	54.665	68.650	0.942	0.000	72.594	1.000
10%	4.503	5.142	n/a	56.313	71.755	0.982	0.000	73.384	1.000
15%	5.327	6.003	n/a	57.579	74.117	1.012	0.000	74.366	1.012
20%	6.138	6.808	n/a	58.623	76.150	1.038	0.000	76.150	1.038
25%	6.923	7.596	n/a	59.588	78.052	1.064	0.000	78.052	1.064
30%	7.690	8.391	n/a	60.418	79.942	1.088	0.000	79.942	1.088
35%	8.492	9.209	n/a	61.210	81.596	1.111	0.000	81.596	1.111
40%	9.363	10.064	n/a	61.952	83.389	1.134	0.000	83.389	1.134
45%	10.237	10.972	n/a	62.630	85.223	1.158	0.000	85.223	1.158
50%	11.184	11.948	n/a	63.313	87.219	1.186	0.000	87.219	1.186
55%	12.252	13.013	n/a	64.012	89.333	1.214	0.000	89.333	1.214
60%	13.456	14.193	n/a	64.680	91.694	1.246	0.000	91.694	1.246
65%	14.796	15.521	n/a	65.412	94.314	1.282	0.000	94.314	1.282
70%	16.338	17.049	n/a	66.175	97.402	1.323	0.000	97.402	1.323
75%	18.147	18.849	n/a	67.053	100.845	1.371	0.000	100.845	1.371
80%	20.465	21.044	n/a	68.018	105.297	1.432	0.000	105.297	1.432
85%	23.353	23.848	n/a	69.074	110.821	1.506	0.000	110.821	1.506
90%	27.397	27.701	n/a	70.355	118.630	1.611	0.040	118.630	1.611
95%	34.254	33.735	n/a	72.050	131.112	1.783	0.126	131.112	1.783
Filter Minimum									
Filter Maximum									
Type (1 or 2)									
# Values Filtered	0	0	0	0	0	0	0	0	0
Scenario #1									
Scenario #2									
Scenario #3									
Target #1 (Value)	0.685	1.530	n/a	49.833	56.196	0.786	0.000	71.221	1.000
Target #1 (Perc%)	0.011%	0.011%	n/a	0.011%	0.011%	0.011%	0.011%	0.011%	0.011%
Target #2 (Value)	68.424	46.711	n/a	78.153	182.040	2.419	0.486	182.040	2.419
Target #2 (Perc%)	99.989%	99.989%	n/a	99.989%	99.989%	99.989%	99.989%	99.989%	99.989%
Target #3 (Value)						1.000	0.535		
Target #3 (Perc%)						12.830%	100.000%		

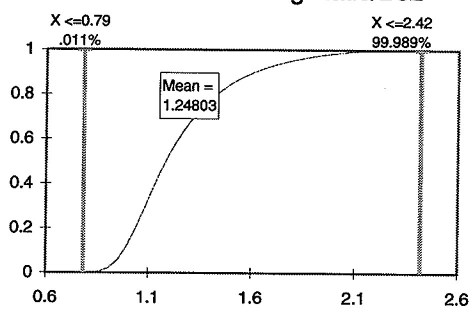
Monte Carlo Simulation Summary for the RRM Aqueduct Section

Simulation Results for Resisting Ratio

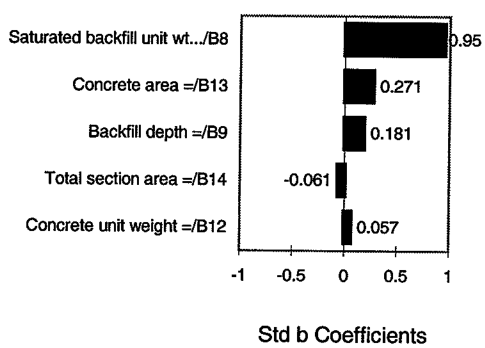
Distribution for Resisting Ratio/B32



Distribution for Resisting Ratio/B32



Regression Sensitivity for Resisting Ratio/B32



Summary Information	
Workbook Name	liability analysis RRM .xls
Number of Simulations	1
Number of Iterations	30000
Number of Inputs	6
Number of Outputs	9
Sampling Type	Latin Hypercube
Simulation Start Time	4/29/2002 18:44
Simulation Stop Time	4/29/2002 18:45
Simulation Duration	00:00:30
Random Seed	1580598996

Summary Statistics			
Statistic	Value	%tile	Value
Minimum	0.767	5%	0.94
Maximum	2.568	10%	0.98
Mean	1.248	15%	1.01
Std Dev	0.258	20%	1.04
Variance	0.066	25%	1.06
Skewness	1.189	30%	1.09
Kurtosis	4.375	35%	1.11
Median	1.186	40%	1.13
Mode	0.908	45%	1.16
Left X	0.786	50%	1.19
Left P	0.01%	55%	1.21
Right X	2.42	60%	1.25
Right P	99.99%	65%	1.28
Diff X	1.63	70%	1.32
Diff P	99.98%	75%	1.37
#Errors	0	80%	1.43
Filter Min		85%	1.51
Filter Max		90%	1.61
#Filtered	0	95%	1.78

Sensitivity			
Rank	Name	Regr	Corr
#1	Saturated backfill unit wt.	0.950	0.930
#2	Concrete area	0.271	0.262
#3	Backfill depth	0.181	0.169
#4	Total section area	-0.061	0.173
#5	Concrete unit weight	0.057	0.061
#6	Saturated ballast unit weight	0.000	-0.011
#7			
#8			
#9			
#10			
#11			
#12			
#13			
#14			
#15			
#16			