The University of Southern Queensland

Faculty of Health, Engineering and Sciences

Future Trend of Retaining Walls in South East Queensland

A dissertation submitted by

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In fulfilment of the requirements of

Courses ENG4111 & 4112 Research Project

Towards the degree of

Bachelor of Engineering (Honours)

Submitted: October, 2017

Abstract

Earth retention systems such as retaining walls are versatile structures used extensively across the civil and construction sectors. Although the methodologies used for wall design can be considered fairly simplistic, wall failures are a surprising common occurrence in the civil construction industry.

This dissertation sought to examine the prominent retaining wall systems used in South-East Queensland and investigate the various theories employed to define their strength requirements. It identifies the bored pier retaining wall as having inherent issues with failure throughout the South East and identifies an appropriate design methodology to undertake theoretical analysis of current practices.

The study shows that there is a lack of unified understanding within the engineering practice regarding element capacities and the lateral earth pressure loadings for bored pier retaining walls. Alternative design iterations used in the industry are compared with the results of the completed design analysis. The results indicate poor performance with regard to soil failure and flexural capacities.

It is hoped the recommendations of this study be considered by regarded members of the engineering profession and lead to a unified understanding and design methodology for bored pier retaining walls. Improvements to the performance of these walls will greatly reduce the failure rates within the civil construction industry.

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Certificate of Dissertation

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged. I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Acknowledgements

I would like to take this opportunity to express my gratitude to the people who mentored and assisted me throughout this project year.

To my USQ supervisor, Dr. Weena Lokuge, for her ongoing support, advice and assistance.

Industry supervisor, Mr Ching Meng Tan, for his industry expertise, willingness to teach and technical mentorship.

And finally to my family, without their support and sacrifice my time at USQ would not have been possible.

Simon Stewart

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1.1 Background

Earth retaining structures, and in particular retaining walls, are a common place and versatile structure used frequently in the construction industry. From household backyards to major infrastructure projects the selection of an appropriate retaining system is critical to achieve the economic, aesthetic and performance requirements set out by clients and design standards alike. With an industry climate that demands value for money, the optimisation of wall design is critical for both the client and the contractor.

Retaining wall design can be complex and requires a multi-disciplinary engineering approach. Das, B (2010) cites that proper design and construction of earth retention structures requires a thorough knowledge of the lateral forces that act between the retaining structures and the soil masses being retained. Further to this, the designing engineer must have a theoretical working knowledge of geotechnical, hydraulic and structural first principals.

The project seeks to identify and examine the prominent retaining wall systems used in South-East Queensland and provide recommendations of best practices. The project attempts to critically analyse and optimise the design of the bored pier retaining wall using both theoretical methodologies and computer software packages.

1.2 Project Aim

The principal aim of this dissertation is to appropriately analyse an earth retention structure and provide industry with recommendations relating to design optimisation and unification of design methodologies used in engineering practice.

Research will primarily investigate the parameters and equations relating to earth retention structure design to provide a solid foundation from which to propose an optimised design solution. Secondly, research will identify and examine the commonly used retaining wall structures in South East Queensland and provide background as to the industry expectations of retaining walls in various industry sectors.

An earth retention structure will be suitably identified to undergo analysis to identify shortcomings in current industry practice and provide industry with reasonable and applicable findings for consideration in the civil construction industry.

1.3 Project Objectives

To achieve the aims of this project the following objectives have been determined.

Objective 1:

The project is to research the key parameters, theories and equations used in retaining wall design to appropriately identify the methodologies to be used in the analysis of a retaining wall.

Objective 2:

Conduct research to identify the earth retention structures commonly utilised in South East Queensland and provide a detailed overview of industry requirements, construction methodologies, advantages and disadvantages and costing associated with each.

Objective 3:

Identify with justification a suitable wall to undergo further analysis using information and findings from the literature review. Complete a structural analysis of the subject wall and provide comment on design iterations currently used in industry practice.

Objective 4:

Provide a design alternative in accordance with relevant Australian Standards using the completed structural analysis with reference to the findings of the design iteration review.

Chapter Two: Literature Review

2.1 Introduction

This chapter discusses a variety of relevant principals, theories and topics that influence the design requirements of earth retention structures. Soil parameters, site investigations and lateral earth pressures are discussed in detail.

2.2 Soils

2.2.1 Introduction

Soil mechanics play a vital role in the performance and subsequent life expectancy of almost all structures designed today. Fang, Y & Bo, L (2016) describes soil mechanics as the basic theory of geotechnical engineering, and its research object is the natural porous geological materials consisting of mineral particles, liquid and gas. Once defined, engineers apply the mechanical properties of soils to a wide variety of engineering problems. "Since ancient ages, engineers have been handling soils as an engineering material for various construction projects" (Isao, I & Hazarika, H 2015). While historical structures such as the Great Wall of China and the Egyptian pyramids relied primarily on accumulated experiences, Isao, I & Hazarika, H (2015) states that during the eighteenth and nineteenth centuries, modern soil mechanic theories such as Rankine and Coulombs theories of lateral pressure were developed. These two theories are of particular importance to the design of retaining walls. Ultimately all structures transfer their load to a soil mass and as such, civil engineers must rely on defined soil parameters to accurately and safely predict the behaviour and relationship between structures and their foundation materials.

2.2.2 Soil Classification

Acquired knowledge of a soil's classification makes it possible for the engineer to estimate and predict soil behaviour and determine potential problems. Al-Khafaji & Andersland (1992) states that by knowing a soil's classification, it is possible to gain insight into a soil's behaviour during construction and under imposed structural loads, while reaffirming that a soil classification system is by no means a substitute for laboratory testing. Classification systems provide a means to define the general characteristics of soils for engineering purposes commonly based on particle size, distribution and plasticity. Soils can be defined as four distinct groups: gravel, silt, sand and clay. Consisting of three phases, solid, liquid (water), and gas (air). Fang, Y & Bo, L (2016) defines soil as an assemblage of non-metallic soil particles.

Soils consist of primarily solid particles that range in size from less than a micron to several millimetres. Many of the engineering aspects of soil behaviour depend on particle size and civil engineers classify soils into gravel, sand, silt and clay. Figure 1 tables the particle sizes pertaining to each soil classification.

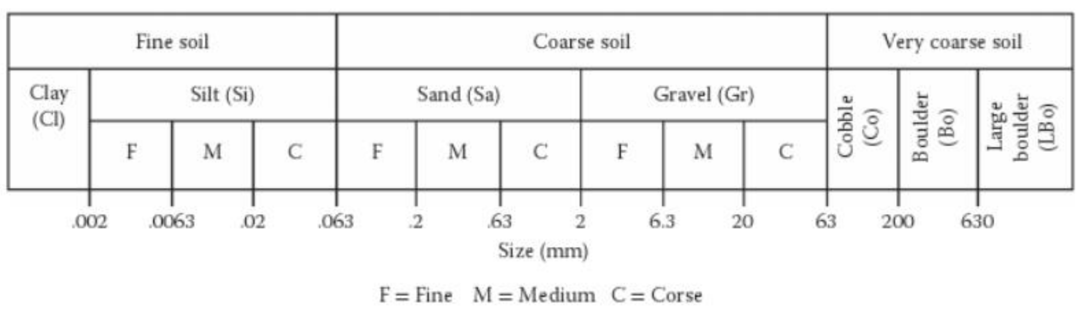

Figure 1.3 Classification of soils according to particle size (size ranges and symbols from EN ISO14688-1: 2002, Geneva, International Organization for Standardization.)

Figure 1: Classification of soils according to particle size (William P, 2013)

The United States Department of Agriculture (USDA) developed a textural classification system based on particle size distribution. Figure 2 shows the method of defining a soil based on particle size distribution. Although a simple, quick and effective method of soil classification, the USDA textural triangle does not take into account the plasticity of soils due to the presence of clay minerals to accurately interpret soil characteristics. Das, B (2010) indicates that the amount of clay minerals present in a soil will dictate to a great extent the physical properties of a soil. It is therefore necessary for an engineer to use a system that accounts for the plasticity of soils.

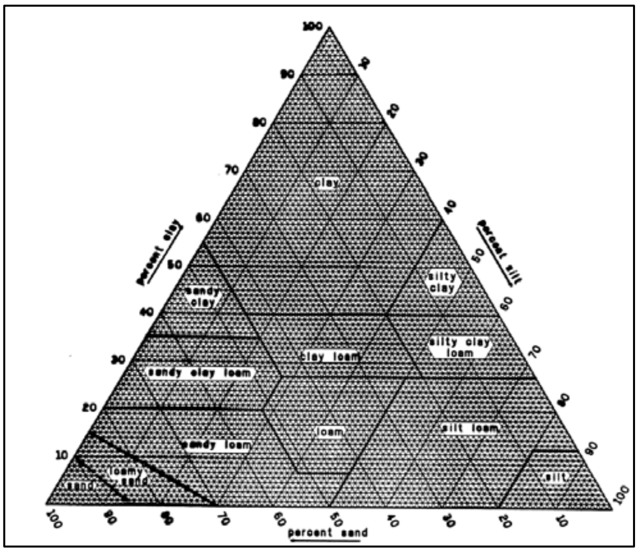

Figure 2: USDA Textural Triangle (USDA 1987)

Two alternative classification systems that account for soil plasticity are the United Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO). These are regularly used in the industry by practicing engineers and are discussed in more detail below.

The Unified Soil Classification System (USCS) was developed by Casagrande in 1942 for the use of airfield construction by Army Corps during World War 2. The information required for proper classification includes:

- Percent of gravel;
- Percent of sand;
- Precent of silt and clay;
- Uniformity Coefficient (Cu);
- Liquid and Plasticity Indexes. (Das, B 2010).

The system firstly groups soils into clays, silts, gravels or sands. Sieve analysis is then conducted with soils that are gravelly and have less than 50% passing of a No. 200 sieve classed as either G (gravels) or S (sands). Soils with greater than 50% passing through the No. 200 sieve are classified as M (inorganic silt), C (inorganic clay), O (organic silts and clay) and Pt (peat and other highly organic soils). Next, plasticity index and further sieve analysis is completed to further define the soil type. On completion, the soil sample will be defined as one of the 35 granular or 36 fine grained soil classifications available.

The American Association of State Highway and Transportation Officials (AASHTO) was developed in 1929 and has undergone multiple revisions since this time. Soils are classified into seven major groups, A1 through to A7. A1 to A3 are granular materials in which 35% of soils will pass through the No.200 sieve. A4 to A7 are fine grained soils which have a greater than 35% passing of the No. 200 sieve. The liquid and plasticity indexes of these soils are required to define A2 and A4-A7. The US Department of Transportation, Federal Highway Administration presents the required figures to determine the classification of soils using either AASHTO or USCS.

2.2.3 Strength of Soils

Masada T (2009) defines the shear strength of a soil as the internal resistance per unit that a soil can offer to resist failure and sliding along any plane inside it. Simply put, it is a soil's ability to sustain load without excessive distortion or failure. An understanding of shear strength in soils is critical to the design of a variety of structures according to Isao, I & Hazarika, H (2015) that cites retaining walls, foundations embankments and other earth structures as requiring a thorough understanding of key soil parameters. The strength of a soil can be described as a function of three parameters, the normal stress applied, the internal angle of friction and cohesion.

The internal angle of friction can be described as the friction between the degrees of interlocking between particles. Whilst dependent on soil characteristics such as soil mineral type, the soils texture, shape and gradation, and void ratio; the internal angle of friction cannot exist without any normal stress acting on the soil mass (Masada 2009).

Cohesion is considered an important factor in a soils consistency. Yokoi H (1968) describes two alternative definitions for cohesion. Firstly, the cohesive force that takes place between adjacent particles. And secondly, the shear strength when compressive forces are equal to zero. The bond formed between soil particles is due to electrostatic attractions, covenant links and cementation.

For design applications the applied normal stress, soil cohesion and the angle of internal friction are required to determine the shear stress using the Mohr-Coulomb Failure Criteria.

$$\tau_f = c + \sigma \tan \phi \tag{1}$$

The preceding equation is referred to as the Mohr-Coulomb failure criterion. In figure 3 below this is physically illustrated as the shear failure envelope plotted on the x-y axis of the Mohr's circle. In reality the shear failure envelope will form a slightly curved line. However Das, B (2010) suggests that for most soil mechanics problems, it is sufficient to approximate the shear stress on the failure plane as a linear function of the normal stress.

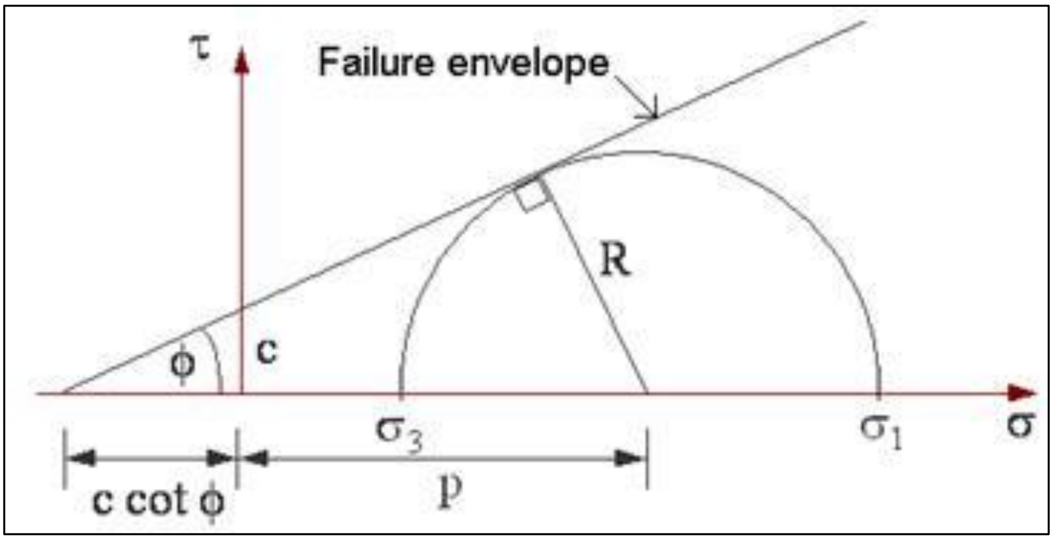

Figure 3: Mohrs circle failure envelope (NPTEL n.d)

2.2.4 Effective Stress

Saturated soils are soils that have their voids completely filled with water. When subjected to a load the stress in a partially or fully saturated soil is carried by both the soil and the water. The stress carried by the water is defined as the pore-water pressure and leads to the concept of effective and normal stress. The effective stress is defined by the normal stress minus the pore-water as described in the below equation

$$\sigma' = \sigma - u \tag{2}$$

where σ' = effective stress, σ = normal stress and u = pore-water pressure.

Das, B (2010) summarised effective stress as the force per unit area carried by the soil skeleton, suggesting that the effective stress concept is probably the most important concept in geotechnical engineering.

Sands and gravels allow water to flow easily through the voids due to their larger particle sizes. As such, sands and gravels are unable to sustain a pore-water pressure for any consequential length of time provided adequate drainage is available. Clays and silts however restrict the movement of water due to their reduced void ratio, allowing a pore-water pressure that is significantly higher than that in surrounding soils to be present for a relatively long period of time. William, P (2013) advises that when subjected to a load such of that of a structure, saturated soils in the loaded area will experience an increase in pore water pressure. As this pressure decreases, stresses are increased on the soil and consolidation occurs. As clays and silts are generally more compressible than sand and gravel, settlement can occur over a long period over time. As such, William, P (2013) suggests that engineers designing the foundations of a building on a clay soil would be “primarily concerned about the possibility of large settlements occurring over a long period of time”.

2.2.5 Bearing Capacity

Foundations are the final structural element before a load path is transmitted into the soil. Das, B (2010) states that the uniform settlement of a structure does not produce cracking; on the other hand, differential settlement may produce cracks and damage to buildings. Therefore, appropriately designed foundations need to transfer the load without overstressing the soil below. Isao, I & Hazarika, H (2015) describes bearing capacity failure as the excessive settlement of foundations without any increase in applied pressure. Due to the nature of soil mechanics and the limitations of mathematical analysis Al-Khafaji, A & Andersland, O (1992) emphasises the importance of applying a large factor of safety (FOS) to avoid bearing capacity failure in foundations.

According to Sivakugun, N & Pachico, M (2010), the use of presumptuous bearing capacities of soils found in various standards can be used as a conservative guide for preliminary design however they do not reflect the site, geological conditions, shear strength or angle of friction. Figure 4 details an example of presumptuous bearing capacities detailed by the British Standards Institution (1986).

TABLE 3.3 Presumed Bearing Capacity Values

Soil Type	Bearing Capacity (kPa)
Rocks	
Hard and sound igneous and gneissic rock	10,000
Hard limestone/sandstone	4,000
Schist/slate	3,000
Hard shale/mudstone or soft sandstone	2,000
Soft shale/mudstone	600–1,000
Hard sound chalk or soft limestone	600
Granular soils	
Dense gravel or sand/gravel	>600
Medium-dense gravel or sand/gravel	200–600
Loose gravel or sand/gravel	<200
Dense sand	>300
Medium-dense sand	100–300
Loose sand	<100
Cohesive soils	
Very stiff clays	300–600
Stiff clays	150–300
Firm clays	75–150
Soft clays and silts	<75

After BS8004:1986 (British Standards Institution 1986) and Canadian Geotechnical Society (1992).

Figure 4: Presumed bearing capacity properties of soils (Sivakugan N & Pachico M 2010)

2.2.6 Bearing Capacity of Foundations

Foundations can be categorised into two groups; shallow foundations and deep foundations.

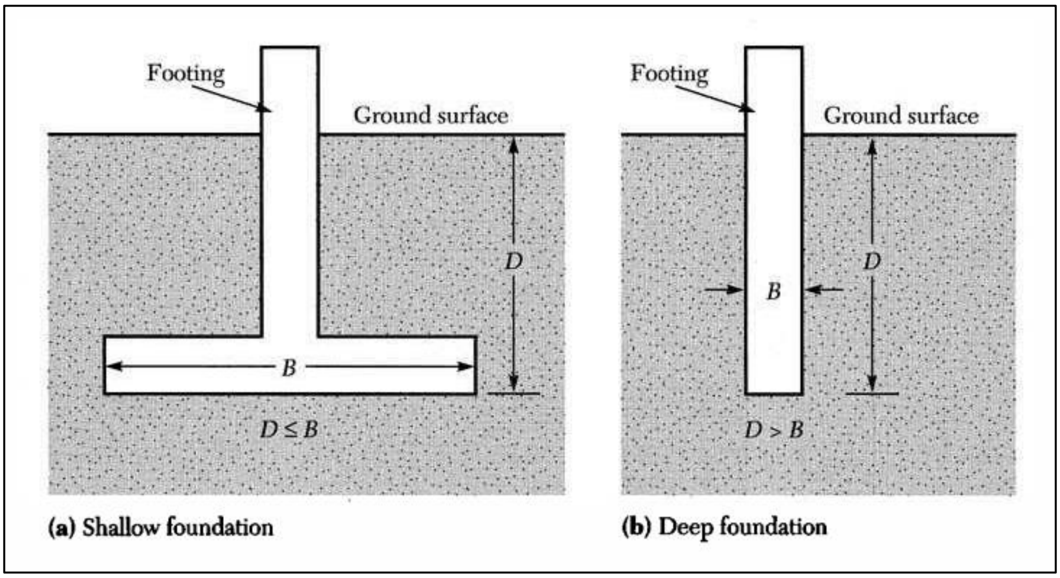

Figure 5: Shallow and deep foundations (Al-Khafaji, A & Alderson, O 1992)

It is common practice to define shallow foundations as a foundation where the depth (*Df)* is less than the breadth *(B).* Sivakugun, N & Pachico, M (2010) advises that shallow foundations are generally designed to satisfy two criteria: bearing capacity and settlement. Shallow foundations will generally have a large load-bearing area which enables the distribution of loads such as those from columns. Failure mechanisms of shallow foundations due to inadequate bearing capacity can be seen in figure 6 below; general shear, localised shear and punching shear.

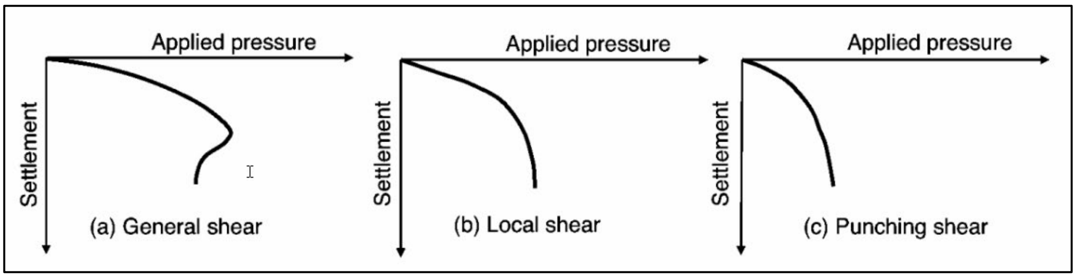

Figure 6: Three failure mechanisms of bearing capacity failure in soils (Sivakugun, N & Pachico, M 2010)

The two common theories for shallow foundation bearing capacity are Terzaghi Ultimate Bearing Capacity and the Myerhof Bearing Capacity. Terzaghi's theory is discussed further below.

In 1943, Terzaghi developed a theory of bearing capacity for shallow soils based on a previous model developed by Prandtl (1921). It involves several assumptions including:

- Uniform soil;
- $Df \leq B$;
- Negligible friction and cohesion forces acting at the sides of the foundation;
- Water level is below zone II;
- Concentric and vertical loading.

(Al-Khafaji, A & Andersland, O 1992).

The theory is based on the development of three zones of plastic equilibrium after *general shear failure* of the soil beneath the footing. As the ultimate capacity load is applied to the footing, general shear failure occurs and subsequent passive failure acts upon the soil wedge. This causes heaving to be present at the surface adjacent to the foundation. It is noted that *local shear failure* will occur in loose-medium dense sands. With reference to figure 7, the slip zone will end inside the radial shear zone.

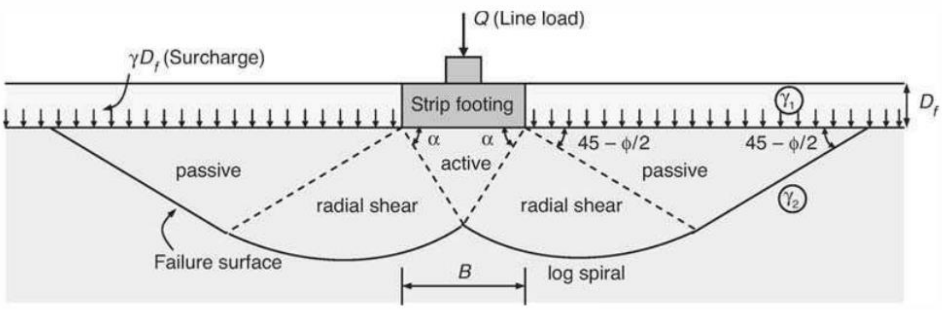

FIGURE 3.7 Assumed failure surfaces within the soil during bearing capacity failure.

Figure 7: Terzaghi's bearing capacity failure zones (Sivakugun, N & Pachico, M 2010)

Terzaghi's bearing capacity equation is defined as

$$q_u = q_c + q_q + q_\gamma = c'N_c + qN_q + \frac{1}{2}\gamma B q N_\gamma \tag{3}$$

where N_c, N_q and N_γ are bearing capacity factors dependent on the soil friction angle.

Krizek (1965) proposed approximations for the values of N_c, N_q *and* N_γ (deviation of 15%) for soils with a friction angle (ϕ) of 0 - 35°:

$$N_c = \frac{228+4.3\phi}{40-\phi} \tag{4}$$

$$N_q = \frac{40+5\phi}{40-\phi} \tag{5}$$

$$N_\gamma = \frac{6\phi'}{40-\phi'} \tag{6}$$

The table below presents Terzaghi's equations for a number of alternative foundation arrangements.

Table 10.1 Terzaghi's Ultimate Bearing Capacity Equations

Type of Shallow Footing	General Shear Failure	Local Shear Failure
Long $B \times \infty$	$q_{ult} = cN_c + qN_q + \frac{1}{2}\gamma BN_\gamma$	$q_{ult} = \frac{2}{3}cN'_c + qN'_q + \frac{1}{2}\gamma BN'_\gamma$
Rect-angular $B \times L$	$q_{ult} = \left(1 + 0.3\frac{B}{L}\right)cN_c + qN_q + 0.4\,\gamma\,BN_\gamma$	$q_{ult} = \left(1 + 0.3\frac{B}{L}\right)cN'_c + qN'_q + 0.4\,\gamma\,BN'_\gamma$
Square $B \times B$	$q_{ult} = 1.3\,cN_c + qN_q + 0.4\,\gamma BN_\gamma$	$q_{ult} = 1.3\,cN'_c + qN'_q + 0.4\,\gamma BN'_\gamma$
Circular Diam. $= D$	$q_{ult} = 1.3\,cN_c + qN_q + 0.3\,\gamma BN_\gamma$	$q_{ult} = 1.3\,cN'_c + qN'_q + 0.3\,\gamma BN'_\gamma$
	Find N_c, N_q, and N_γ using Figure 10.4 with ϕ.	Find N'_c, Nk'_q, and N'_γ using Figure 10.6 with $\phi = \tan^{-1}(\frac{2}{3}\tan\phi)$.

Figure 8: Terzaghi's equations of ultimate bearing capacity (Sivakugun, N & Pachico, M 2010)

2.2.7 Site Investigations

Site investigations are considered to be one of the most critical aspects of any project. As earth retention structures rely so heavily on the accuracy of soil properties it is crucial that field investigations produce accurate results. Standards Australia (2017) recommends that the delivery of geotechnical site investigations should follow an iterative process in which the outcomes of the investigations are reviewed against the purpose for which the investigation is being carried out and further investigations are planned as required. This is discussed in more detail within the standard. Mohamed et al (1998) advocates that there seems to be a belief that the larger the amount of data produced during a field investigation, the greater the probability the project will be completely successful without complication or delay.

Figure 9 displays the required parameters to be defined by a site investigation where retaining walls are to be constructed. In order to define soil parameters, field work is required and typically includes:

- Mapping of the topography, geology, geomorphology and other relevant features;
- Logging of cuttings or other exposures;
- A program of sub-surface works (such as boreholes, test pits and probe tests such as cone penetration tests);
- Measurements, e.g. recording of groundwater levels and in situ testing;
- Collection of soil, rock and groundwater samples for subsequent testing.

(Standards Australia 2017)

Isao, I & Hazarika, H 2015 indicates that two soils obtained by closely separated boreholes can suggest vastly different soil types. Reinforcing the requirement for both insitu and laboratory tests on soil specimens as being critically important to define index parameters and engineering characteristics.

TABLE 2.1

INVESTIGATION CONSIDERATIONS

Investigation considerations	Structure classification		
	C	B	A
Substrata type	●	●	●
Effect of drainage discharge onto surrounding site	●	●	●
Nature of retained material	●	●	●
Site topography	●	●	●
Foundation and embankment strength parameters	●	●	○
Existing ground water levels and seepage	●	●	○
Effect of excavations or filling	●	●	○
Location of existing or proposed adjacent structures	●	●	○
Effect of modified water table on surrounding site	●	○	○
Global stability	●	○	○
Impact of structure 'zone of influence'	●	○	○
Ground movement	●	○	○

● denotes detailed site-specific information required.

○ denotes only general consideration of the item required.

Figure 9: Suggested parameters to be determined during site investigations (AS4678 Australian Standard 2002)

2.3 Lateral Earth Pressure

2.3.1 Introduction

Earth pressure is the lateral pressure exerted by the soil on a shoring system and is perhaps the most critical parameter relating to retaining wall design. In order to design earth retaining structures such as retaining walls, it is necessary to determine the magnitude of the lateral pressures to which the structure is subjected. Al-Khafaji, A & Andersland, O (1992) defines earth pressure as the force per unit area of the soil on a structure. Its magnitude depends on the physical properties of the soil, the nature of the soil structure interface and possible modes of deformation of the structural system. They conclude that the analysis and design of retaining structures such as walls, cofferdams, basement walls, and bulkheads require a thorough knowledge of the lateral forces acting between the structure and the soil mass they help support. Due to many variables it is essential that good engineering judgement be used. Various earth pressure theories assume that soils are homogeneous, isotropic and horizontally inclined leading to hydrostatic or triangular pressure distributions.

The following sections discuss methods for the calculation of lateral earth pressures including Rankine and Coulomb active and passive case theories.

2.3.2 At-Rest

When a wall is not allowed to move, the stresses at a particular depth are under elastic equilibrium with no shear stress. A wall in this state is considered to be subject to at-rest lateral earth pressure.

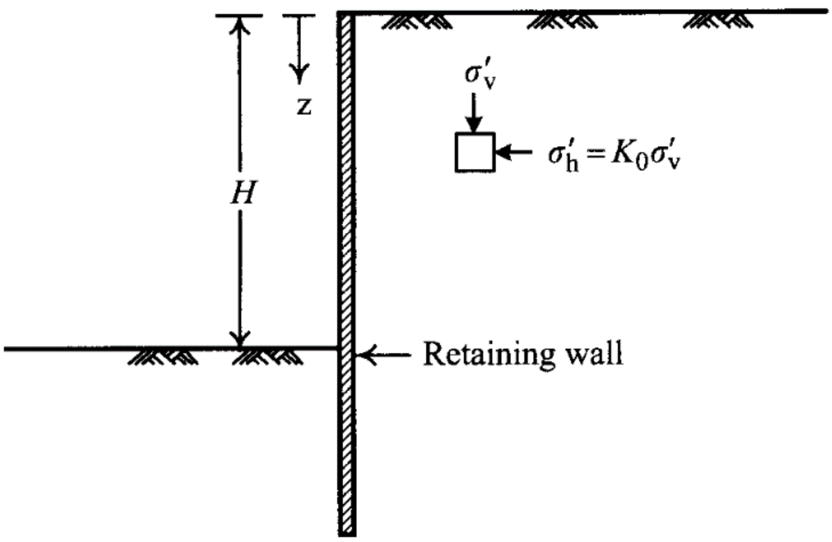

Figure 4.1 Lateral earth pressure at rest.

Figure 10: At rest state of a retaining structure (Boeraeve, I 2008)

Referring to figure 10 we can determine the horizontal effective stress, σ'_h and horizontal effective stresses using the formulas:

$$\sigma'_v = K_0\, \sigma'_v \qquad (7)$$

$$\sigma'_h = K_0\, \sigma'_v \qquad (8)$$

where, K_0 is the coefficient of lateral earth pressure at rest.

The total stress can be calculated as:

$$\sigma_h = \sigma'_h + u \quad (9)$$

where: u is pore-water pressure; and

K_0 is the at-rest earth pressure coefficient

For cohesionless soils K_0 can be calculated using Jaky's equation:

$$K_0 = 1 - sin\phi' \quad (10)$$

where: ϕ' is the effective internal angle of friction

Das, B (2010) suggest that when designing a wall that may be subjected to lateral earth pressure at rest, one must take care when evaluating the value of K_0. Evidence from laboratory tests conducted by Sherif et al. (1984) suggest that equation 10 may grossly underestimate lateral earth pressure at rest for dense, compacted sands.

For cohesionless soils in pre-consolidated or unloading states (overconsolidated), K_0 can be expressed by the following equation:

$$K_{0,OC} = K_{0,NC} OCR^{\alpha} \quad (11)$$

where: $\alpha \approx sin\phi'$; and

$$OCR = \frac{\sigma'_c}{\sigma'_v} \quad (12)$$

Generally speaking, for normally consolidated soils, equation 10 produces satisfactory results however for overconsolidated soils equation 11 results are relatively inaccurate and therefore an situ test should be carried out to obtain the most accurate value of K_0.

For cohesive soils, the coefficient of earth pressure at rest can be approximated in terms of the angle of internal friction, ϕ and the overconsolidation ratio, OCR. This can be given as:

$$K_0 = (0.95 - sin\phi')\sqrt{OCR} \quad (13)$$

2.3.3 Rankine's Earth Pressure Theory

Scottish engineer and physicist, William Rakine (1857), developed the Rakine theory of lateral earth pressure in conditions of failure in front of and behind a retaining wall on the basis of plastic equilibrium (condition where every point in a mass is on the verge of failure). Rankine's theory assumes a smooth, frictionless vertical wall and plane failure surfaces. This wall state implies there are no shear stresses acting on horizontal and vertical planes and therefore the horizontal and vertical stresses are principal stresses (Al-Khafaji, A & Andersland, O 1992).

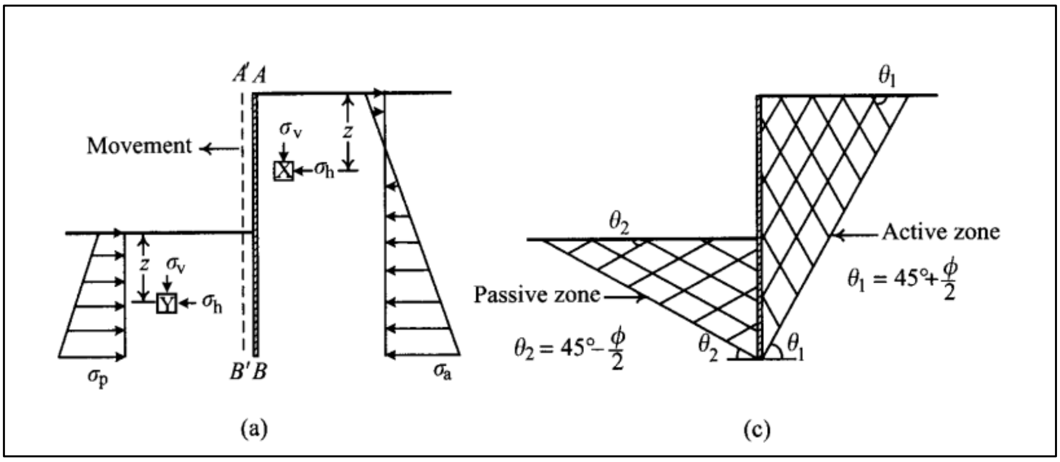

Figure 11: Displacement of a retaining wall and the passive and active zones produced by the movement (Boeraeve, I 2008).

2.3.3.1 Rankine's Active Pressure

As shown in figure 11 above, as the wall moves away from the soil mass gradually the horizontal principal stresses in the soils behind the wall decrease. In this active case, σ'_v is the minor principal stress and σ'_h is the major principal stress.

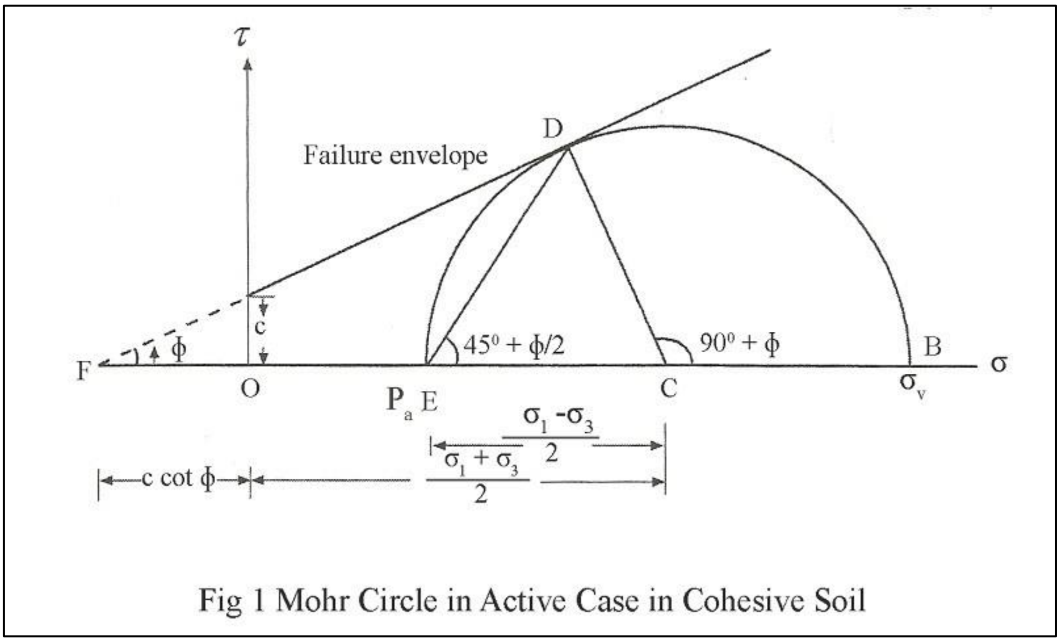

Figure 12: Mohrs circle of Rankine active state for a cohesive soil (Best Engineering Projects 2013)

The effective horizontal stress can be derived either graphically or analytically from the Mohrs circle of Rankine's active earth pressure, numerically represented as

$$\sigma'_h = tan^2\left(45° - \frac{\phi'}{2}\right)\sigma'_v - 2c(tan45° - \frac{\phi'}{2}) \qquad (14)$$

Rankine's coefficient of active lateral pressure is defined as

$$K_a = tan^2(45° - \frac{\phi'}{2}) \qquad (15)$$

Substitution of (7) into (8) gives

$$\sigma'_h = K_a\sigma'_v - 2c\sqrt{K_a} \qquad (16)$$

Note that for cohensionless soils, that is $c = 0$ the horizontal effective pressure is derived as

$$\sigma'_h = K_a\sigma'_v \qquad (17)$$

Rankine's active earth pressures can be calculated for both cohesive and cohesionless soils. These are graphically illustrated in figure 13 below and it can be observed that cohesive soils have a negative horizontal effective stress to a depth of z_0 . Al-Khafaji, A & Andersland, O (1992) advises that the earth pressure in cohesive soils is generally calculated using the positive stress only as tension cracks will form between the soil and wall above the depth z_0.

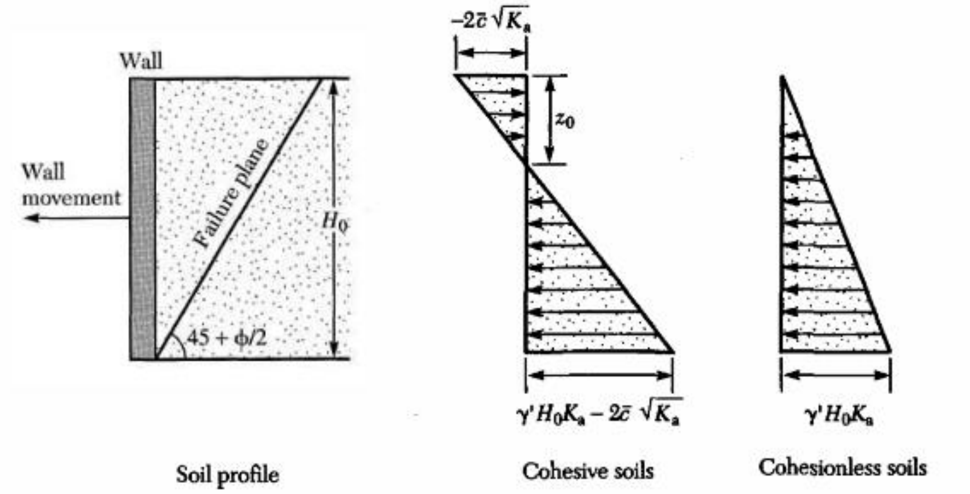

FIGURE 9.4 Graphical illustration of the Rankine active earth pressure acting on a frictionless wall.

Figure 13: Rankine's active earth pressure (Al-Khafaji, A & Andersland, O 1992)

2.3.3.2 Rankine's Passive Pressure

As the wall moves towards the soil mass the horizontal effective stresses increase. This is represented on the Mohrs circle in figure 14. Rankine's passive state can be derived in a similar manner to the active case, however in the passive case σ'_h is the major principal stress and σ'_v, is the minor principal stress. The effective horizontal stress can be derived either graphically or analytically from the Mohrs circle.

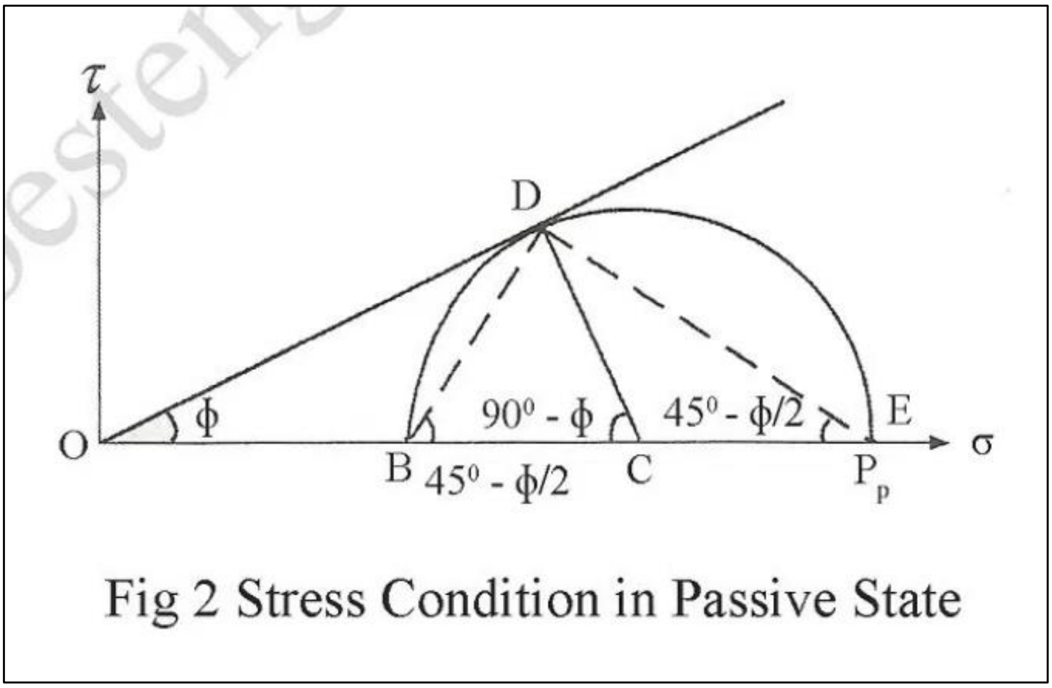

Figure 14: Mohrs circle of Rankine passive state (Best Engineering Projects 2013)

Numerical representation of the Mohrs circle can be defined as

$$\sigma'_h = tan^2\left(45° + \frac{\phi'}{2}\right)\sigma'_v - 2c(tan45° + \frac{\phi'}{2}) \tag{18}$$

Rankine's coefficient of active lateral pressure is defined as

$$K_p = tan^2(45° + \frac{\phi'}{2}) \tag{19}$$

Substitution of (7) into (8) gives

$$\sigma'_h = K_p\sigma'_v + 2c\sqrt{K_p} \tag{20}$$

Note that for cohensionless soils, that is $c = 0$, the horizontal effective pressure is derived as

$$\sigma'_h = K_p\sigma'_v \tag{21}$$

The passive lateral earth pressure for a frictionless wall can be calculated in accordance with figure 15. It can be observed that for both cohesive and cohesionless soils the earth pressure is positive.

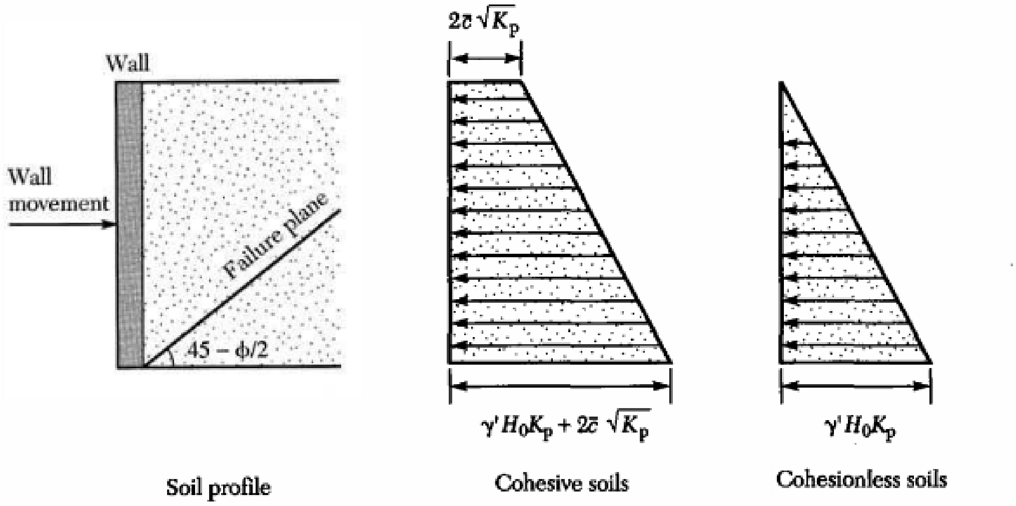

FIGURE 9.5 Graphical illustration of the Rankine passive earth pressure acting on a frictionless wall.

Figure 15: Rankine's passive earth pressure force diagram (Al-Khafaji, A & Andersland, O 1992)

2.3.4 Coulomb Theory

Over 200 years ago Coulomb (1776) presented a theory for passive and active earth pressures against retaining walls. The theory provides a method of analysis that gives the resultant horizontal force on a retaining system for any slope of wall, wall friction, and slope of backfill. This theory is based on the assumption that soil shear resistance develops along the wall and failure plane.

2.3.4.1 Coulomb's Active Pressure

For a retaining system with a continuous sloping granular backfill at an angle, α Coulomb's active pressure can be described using figure 16 below. The probable failure wedge ABC involves the following forces:

- W = weight of the soil wedge;
- F = Resultant of the shear and normal forces on the surface of failure;
- P_a = Active force per unit length of wall. (Das, B 2011)

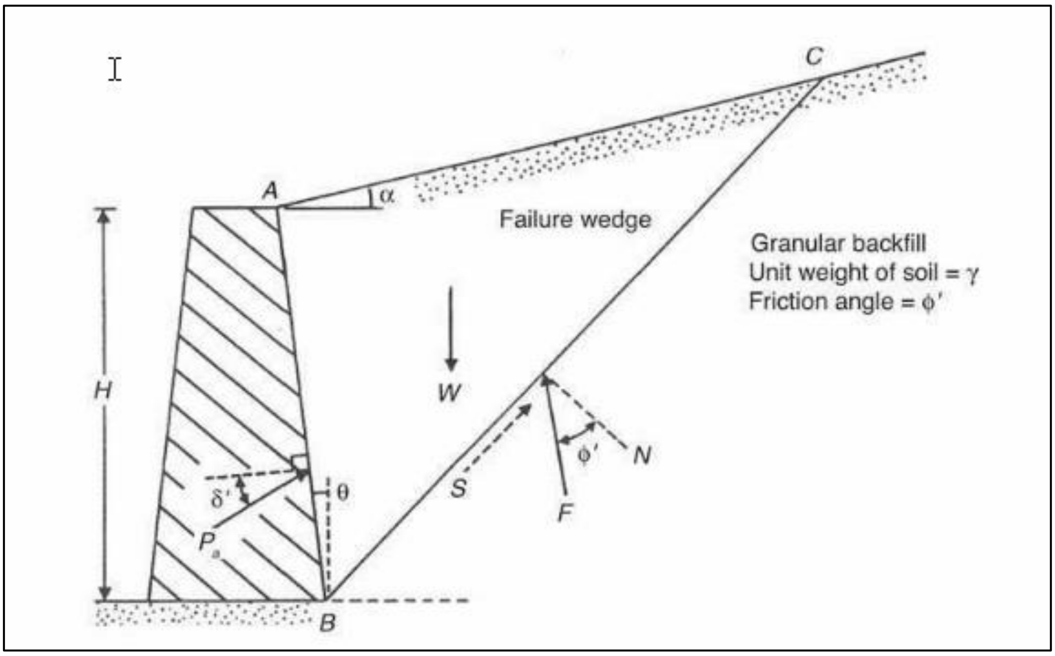

Figure 16: Force diagram of Coulombs' active earth pressures (Das, B 2011)

Coulombs active earth pressure can be derived by the equations below:

$$P_p = \frac{1}{2}\gamma H^2 k_a \tag{22}$$

where ka is calculated by:

$$K_a = \frac{cos^2(\emptyset+\theta)}{cos^2\theta\cos(\delta-\theta)\left[1-\sqrt{\frac{\sin(\emptyset+\delta)\sin(\emptyset+\beta)}{\cos(\delta-\theta)\cos(\beta-\theta)}}\right]^2} \tag{23}$$

2.3.4.2 Coulomb's Passive Pressure

The passive pressure can once again be considered in a similar fashion to that of the active pressure.

Passive pressure, P_p, can be derived by the formula given below

$$P_p = \frac{1}{2}\gamma H^2 K_p \tag{24}$$

Where Kp is calculated as

$$K_p = \frac{cos^2(\phi+\theta)}{cos^2\theta\cos(\delta-\theta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\cos(\delta-\theta)\cos(\beta-\theta)}}\right]^2} \tag{25}$$

2.4 Conclusion

The review of soils and lateral earth pressure has provided a solid foundation in the requirements for determining the loadings experienced by walls and the strength parameters required to resist these loadings in accordance with Australian Standards.

The following conclusions can be drawn:

- Geotechnical surveys generally do not present enough reliable data to conclusively determine the soil parameters at the location of walls on larger sites. It is therefore necessary for the design engineer to attend site to confirm that the insitu soil conditions are suitable.
- Rankine and Column theories are both relied upon within the construction industry and accepted by Australian Standards. It can be said that Rankine produces a more conservative value of lateral earth pressure due to the assumption that a wall is vertical and therefore it is the recommendation of this study that for bored pier retaining walls, Rankine's theory of lateral earth pressure is preferred.
- The shear strength of soils is greatly affected by the presence of groundwater. As such, the design engineer must take care when predicting the active pressures experienced by a wall in regard to pore-water pressure, the unit weight of saturated soils and the effective cohesion and internal angle of friction associated with undrained soils. Likewise, the capacity of passive resistance needs to be evaluated on this same basis.

Chapter Three: South East Queensland Retaining Walls

3.1 Wall Selection

Retaining wall selection depends on a number of factors ranging from performance requirements to the client aesthetic expectations. "Choosing the wrong type of wall may not provide sufficient restraint, may be impractical to construct, and may cause instability in the existing geotechnical conditions" (Hart, J 2013). The following parameters must be considered to provide an effective and efficient design solution:

- Strength;
- Durability or design life;
- Budget;
- Aesthetics;
- Geological conditions;
- Existing services;
- Site conditions.

By ensuring the suitable weighting of the above criteria an effective wall selection can be made.

3.2 Industry Expectations

Commonly considered a 2nd tier structure the industry requirements for retaining walls, particularly for those civil land developments, are primarily based on cost. It is therefore of little surprise that wall failures in this industry are increasingly common. A key theory of this study is that the restrictions and pressures placed on engineers to design walls that can be competitively priced for design and construction subcontracts is a contributing factor underestimation of wall capacity and subsequent failures. The hard dollar culture of this industry is not conducive for the provision of quality.

3.3 Common Retaining Wall Systems

Retaining wall systems used in today's urban development environment can vary greatly. Hart, J (2013) suggests that typically several conditions from the geotechnical, structural, civil, and construction perspectives have a bearing on the type of wall best suited for a particular site. As a result of this theory, it is critical to base the selection of an appropriate wall suitable to economically overcome earth pressures likely to be encountered with reference to the soil conditions identified by geological surveys. This section aims to provide a detailed overview of the common walls used in the South East Queensland civil construction industry. It will discuss the advantages and disadvantages of the wall system, the construction methodologies and materials, the design principals of each wall and provide accurate costings. The retaining structures can be broadly categorised into three categories; cantilever, gravity and mechanically stabilised earth. This study will consider cantilever and gravity walls only.

3.3.1 Cantilever walls

Cantilever wall are a versatile and highly common retaining structure used in a number of construction industries throughout South East Queensland. As their name eludes, cantilever walls act on the principle of a structure fixed at one end. There are a variety of cantilever walls available in the market and these can be further categorised into two distinct types; shallow foundations and deep foundations.

3.3.2 Shallow Foundation Cantilever Walls

Shallow foundation cantilever walls use a spread foundation to transfer the lateral loads of the supported soils to the foundation strata via a combination of dead weight and structural resistance. They are generally constructed of reinforced cement concrete (RCC) and can be found across a variety of industries including civil, marine, commercial and residential.

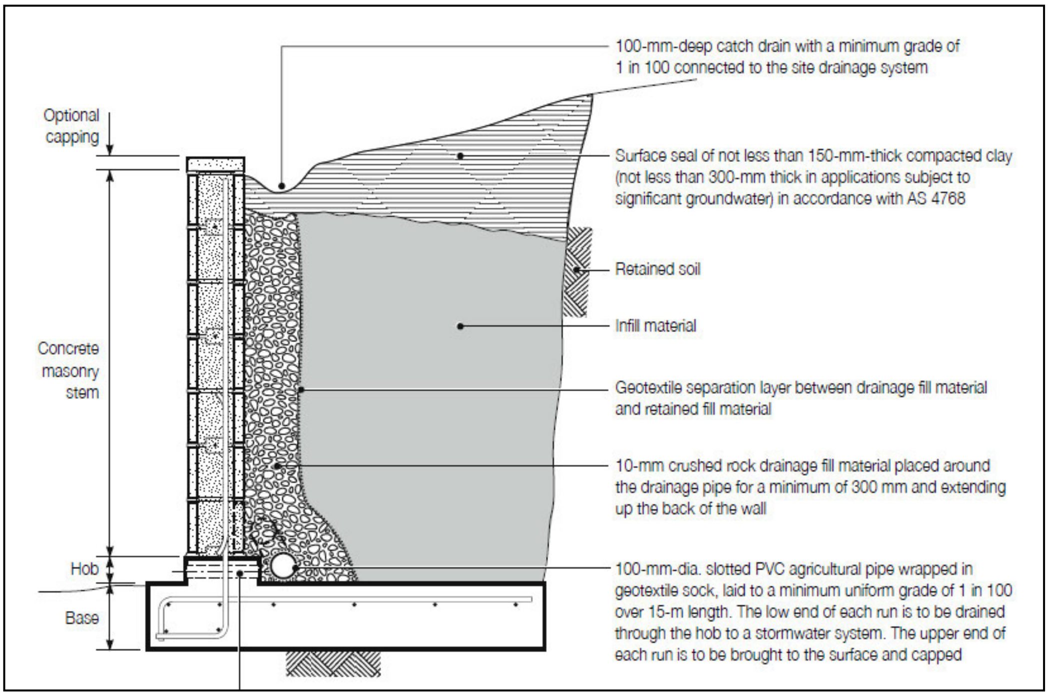

Figure 17: Typical detailing of a reinforced concrete retaining wall with masonry face (CMAA 2013)

3.3.2.1 Characteristics

Advantages of this wall type:

- Less material than gravity walls;
- High aesthetic properties can be achieved due to unlimited finish potential;
- No tieback or soil reinforcement necessary for up to 6m;
- Simple construction staging;
- Minimal impact on space upon completion as the foundation is below the finished ground level.

Disadvantages associated with this wall type:

- Boundary issues due to foundation width;
- Space required behind the wall for foundation (depth of wall cut);
- Excavation and reinforcement requirements for foundations can lead to an extended construction time;
- Susceptible to bearing failure in poor soils;

- Multiple skilled trades required for construction.

3.3.2.2 Construction Methodology - Masonry Block Wall

The following section sets out the general methodology for the construction of a masonry block reinforced concrete cantilever wall on a spread foundation. The methodology covers the trades, materials, plant requirements and general construction procedures. It is assumed that site access and soil conditions are favourable and the wall is being constructed against a cut embankment. Therefore the requirement to import and compact fill in order to achieve site design elevations is not considered. It does however consider the backfilling requirement to satisfy drainage design elements of the wall.

Trades, Material and Plant

Trades
Surveyor
Earthworks contractor
Steel fixer
Concreter
Carpenter / form worker
Block layer
Concrete truck operator
Concrete line / boom pump operator/s

Materials
Formwork and form shutters (if required)
Steel reinforcement
Reinforcement placing accessories (chairs, tie-wire etc.)
Concrete (20/20/80)
Concrete (20/10/180)
Mortar (M4 mortar mix - recommended)
Masonry Blocks '200 series'
100mm un-socked agricultural pipe

A24 geo-synthetic fabric material

Drainage gravel (20mm)

Backfill capping clay

Plant

Excavator

Tipper Truck

Line / Boom Pump

Concrete Truck

Delivery Vehicles

Construction Procedure

1. Site Establishment

Site establishment is critical to achieving efficient construction practices. Critical elements include defining the work zone, locating appropriate lay-down areas and establishing legislative site amenities. Note that in many cases retaining wall contractors are working under minor works subcontracts and amenities are provided by the head contractor in most instances. The contractor is to ensure that all services are clearly marked prior to excavation commencement and that dial before you dig plans (DBYD) have been collected and assessed. In some instances excavation permits are required to be attained from the head contractor. It is critical where the wall is greater than 1m in height that the retaining structure drawings have been designed and certified by a registered engineer.

2. Survey and Alignment

A registered surveyor is to be engaged to provide the required mark out for the perimeter of the wall. This will generally include marker stakes indicating the finished face of the wall and start/finish points. The surveyor should provide a site datum to allow the finished reduced levels (top and bottom) to be confirmed.

3. Foundations

The contractor is to off-set the surveyor markers where construction activities may cause them to be lost. The installation of hurdles/profiles can assist to maintain the surveyors mark out and eliminate the requirement of the surveyor to reinstate the points. The foundations should be measured using the front face of the wall as a guide and dimensioned in accordance with the structural plans. Spray paint or 'dazzle' can be an effective way of marking out the soil for excavation.

4. Excavation

The earthworks contractor is to excavate the required foundation materials as marked and in accordance with the structural drawings. A spotter with a laser level or dumpy should assist the operator to insure that levels are accurately cut and that the achieved dimensions are correct. An accurate excavation will save time, concrete and/or crusher dust when forming and pouring the foundations.

5. Formwork

If required, the contractor is to form the required step and edges of the foundation. If over excavation is encountered, crusher dust can be spread and compacted to reduce the over-spend on concrete.

6. Steel Placement

On completion of the formwork the steel is to be fixed in place in accordance with the structural design. It is important to note that the exposed starters must be capped to prevent injury.

WITNESS POINT - The certifying engineer is required to attend site to ensure that the construction activities up to this point is in accordance with the intended design. In particular the engineer will be ensuring that the soil condition, reinforcement placement and general site parameters are consistent with that of the design. Concrete shall not be poured until the engineer has given approval.

7. Concrete - Foundations

Concrete is to be measured and delivered to site. It is critical that the concrete is ordered, delivered and poured in accordance with the structural specifications. Important considerations include the concrete mix, compaction requirements, delivery waiting time and finish. Note that in many instances a line or boom pump may be required in order to efficiently deliver concrete to the foundation location.

8. Masonry

The block layer is to use the surveyor stakes or off-set profiles to determine the alignment and start/finish points of the wall. The reduced levels must be determined and achieved within the required tolerances. Reinforcement is to be installed concurrently with the masonry blocks by the block layer. Clear out masonry blocks are to be used on the first row to allow for the removal of mortar and other debris collected during the laying of the wall prior to core-filling.

WITNESS POINT - The certifying engineer may wish to attend site to ensure that the blocks, foundations and reinforcement arrangement are adequately constructed and consistent with the design. In particular the engineer will be inspecting the reinforcement placement. Concrete shall not be poured until the engineer has given approval.

9. Concrete - Core-fill

The clear out block is to be reinstated with either formwork or the original 'knockout blocks'. The blocks are to be filled with the prescribed concrete product (high slump mix) to ensure no voids are present on completion. A line or boom pump is generally required to complete this activity.

10. Drainage and Backfill

The installation of the drainage requirements defined by the engineer is critical to prevent excessive pore-water pressure from impacting on the wall. The general methodology and arrangement is as follows:

- Run the specified geosynthetic fabric on the soil side, from the base of the wall to the top of the cut for the entire length of the wall and fix temporarily in place;
- Provide 100mm of drainage material between the wall and the geosynthetic fabric;
- Install 100mm agricultural pipe along the length of the wall and connect or exit the wall at the location specified;
- Continue backfilling the wall with the 20mm drainage gravel stopping 300mm from the top or as described in the construction drawings and fold the geosynthetic fabric over the drainage gravel;
- Compact a clay soil in the form of a water resistant plug to complete the backfill. The clay layer acts to prevent the ingress of surface water behind the wall which may contribute to pore-water pressures.

12. Site Demobilisation

On completion of the project the site is to be cleared and left in an appropriate state for further construction activities to proceed if required. The head contractor or client should be notified of the loading limitations of the wall to ensure that these are not exceeding during construction activities. Important client information includes the zone of influence of the wall both in front and behind. Common mistakes include using vibratory rollers above the wall and excavation of service trenches in front of the wall.

3.3.2.3 Cost Estimate

A worked estimate example is provided below for a reinforced masonry block wall on a spread foundation. Quantities of materials and labour are a typical representation of a 2m high and 50m long retaining wall. Site access is considered to be adequate with preferable soil conditions. Construction methodologies and plant are considered to be consistent with the construction procedures as previously described. Unit rates for materials and labour have been determined through consultation with various local trades, suppliers and quantity surveyors and represent a fair assessment of the local (SEQ) construction climate at the time of this study. An overall rate of $338.36/m2 has been determined for masonry wall construction of the defined magnitude.

Activity	Description	Unit	Rate ($)	QTY (Lm)	Total ($)
1	**General**				
1.001	Survey/Set Out	ea	450	0.02	9.00
2	**Materials**				
2.001	Steel reinforcement (cut/bent)	kg	1.83	31.60	57.83
2.002	Concrete (20MPa foundation)	m3	165	0.72	118.80
2.003	Concrete (20Mpa blockfill)	m3	185	0.21	38.85
2.004	200 series masonry blocks	ea	2.31	25.00	57.75
2.005	100mm unsock agg drain	Lm	4.5	1.00	4.50
2.006	Recycled drainage gravel (20mm)	tne	25	0.80	20.00
2.007	A24 Geosynthetic fabric (2000x50m)	Lm	2.54	1.00	2.54
3	**Plant**				
3.001	6.5t Excavator (cut, foundation and backfill - wet) 3d	day	960	0.06	57.60
3.002	Line Pump x 2	day	750	0.04	30.00
4	**Labour**				
4.001	Supervision	day	600	0.01	6.00
4.002	Set out and excavation supervision 2d x 1pp	day	400	0.04	16.00
4.003	Reinforcement placement (foundation) 2d x 3ppl	day	400	0.12	48.00
4.004	Concrete place (foundation) 1d x 3pp	day	400	0.06	24.00
4.005	Block labour (including wall reo install)	each	4	25.00	100.00
4.006	Concrete place (core filling) 0.5d x 2pp	day	400	0.02	8.00
			Sub-Total (Lm)		598.87
			Margin and O/H (10%)		59.89
			Contignecy (3%)		17.97
			Total (Lm)		676.72
			Total m2 (ex GST)		**338.36**

Figure 18: Costing sheets of Masonry retaining wall on spread foundation

3.3.3 Deep Foundation Cantilever Walls

Unlike a cantilever wall on spread foundations, the pile foundation cantilever wall, also referred to as an embedded cantilever wall, relies solely on passive resistance in front of the wall to provide stability against lateral earth pressure. Two common iterations of pile foundation walls used in the industry are:

- Sheet pile wall;
- Bored pier sleeper walls.

Cantilever sheet pile walls are constructed by vertically driving prefabricated, interlocking steel sections into the ground. Day, R (1999) refers to sheet piles for the use of permanent and temporary structures up to about 4.5m (self-supporting). They are commonly driven prior to excavation commencing and can be further strengthened by tie backs, braces and/or ground anchors.

Figure 19: shows a typical cantilever sheet pile wall supported by ground anchors (Keyword Suggests n.d)

Bored pier sleeper walls are commonly used in urban development projects. They are constructed using steel beams and columns cast monolithically into a bored pier often referred to as a soldier pile. Timber or precast RCC beams or panels are installed between the piles and act as the face of the wall. Bored pier retaining walls are an economical wall, particularly in soft soil conditions where the bearing capacities of soils are low. The section below identifies the common construction practice and cost estimates of the bored pier retaining wall.

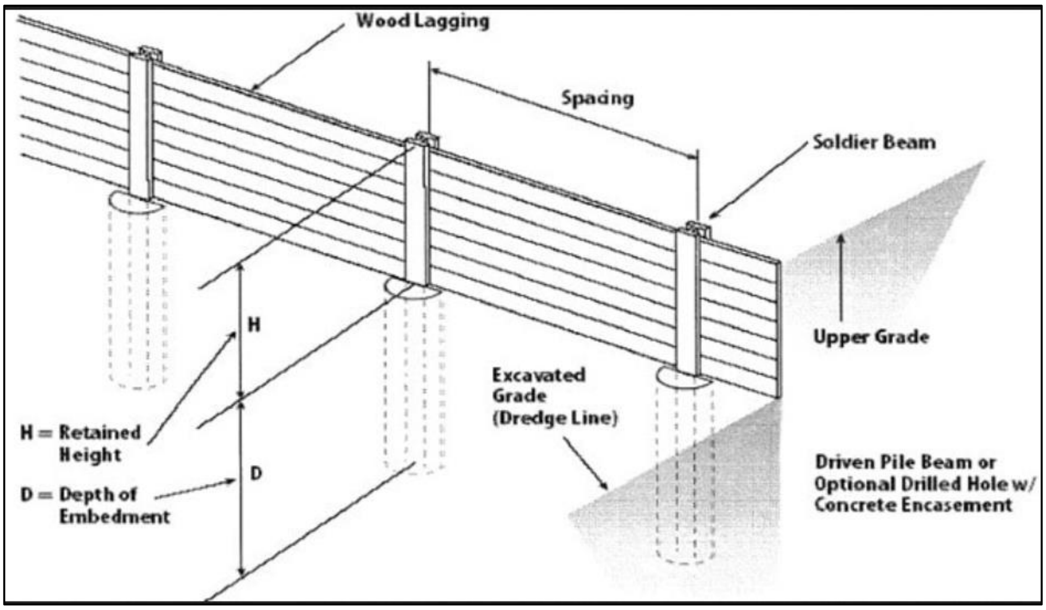

Figure 20: Typical layout of a cantilever wall on bored pile foundations (The Constructor n.d.)

3.3.3.1 Characteristics

The Advantages of this bored pier retaining wall are:

- Narrow;
- Can be installed up to the site boundary with minimal construction backfill required, making them ideal for built up construction areas;
- Fast and relatively simple construction;
- Can be designed to accommodate poor soil conditions;
- Multiple finishes are available for precast panels and sleepers.

Disadvantages associated with this wall type are:

- Specialist equipment required such as auger attachments;
- Cannot be rendered (limited in finishes);
- Difficult to waterproof;

- Can be expensive where rock excavation is required;
- Design variance in industry with failures a common occurrence.

3.3.3.2 Construction Methodology

The following section sets out the general methodology for the construction of a bored pier retaining wall commonly found in civil land developments across South East Queensland. The methodology covers the trades, materials, plant requirements and general construction procedures. It is assumed that site access and soil conditions are favourable and the wall is being constructed against a cut embankment. Therefore the requirement to import and compact fill in order to achieve site design elevations is not considered. It does however consider the backfilling requirement to satisfy drainage design elements of the wall.

Trades Material and Plant

Trades
Surveyor
Earthworks contractor
Carpenter / Installer
Concrete truck operator
Concrete line pump operator

Materials
Concrete (20/20/80)
Concrete Sleepers
Universal Columns and Beams (galvanised)
100mm unsocked agricultural pipe
A24 geosynthetic fabric
Drainage gravel (20mm)
Backfill capping clay

Plant

Excavator

Line Pump

Concrete Truck

Delivery Vehicles

Construction Procedure

1. Site Establishment

Site establishment is critical to achieving efficient construction practices. Critical elements include defining the work zone, locating appropriate lay-down areas and establishing legislative site amenities. Note that in many cases retaining wall contractors are working under minor works subcontracts and amenities are provided by the head contractor in most instances. The contractor is to ensure that all services are clearly marked prior to excavation commencement and that DBYD plans have been collected and assessed. In some instances excavation permits are required to be attained from the head contractor. It is critical where the wall is greater than 1m in height that the retaining structure drawings have been designed and certified by a registered engineer.

2. Survey

A registered surveyor is to be engaged to provide the required mark out for the perimeter of the wall. This generally will include marker stakes indicating the finished face of the wall and start/finish points. The surveyor should provide a site datum to allow the finished reduced levels (top and bottom) to be confirmed.

3. Pier Set-out

The contractor is to off-set the survey markers where construction activities may cause them to be lost. The installation of hurdles/profiles can assist to maintain the surveyors mark out and eliminate the requirement of the surveyor to reinstate the points. The piers are to be marked at their centre along the entire length of the wall set to be drilled using running measurements. Spacings and dimensions of the piers is to be in accordance with the structural design.

4. Excavation

The earthworks contractor is to excavate the piers using a hydraulic auger attachment. A spotter should be present to assist the operator in assuring that a vertical excavation is being achieved and to relocate any large rocks or debris that may otherwise impede the drilling process. It is critical that accurate spacings are achieved in order to minimise the requirement for cutting sleepers between columns and achieving appropriate concrete coverage to the column.

WITNESS POINT - It is best practice for the certifying engineer to attend site periodically during the drilling process to ensure that the soils encountered are consistent with that of the geological surveys. This is a critical element to the successful performance of this wall type. Concrete should not be poured until the engineer has given approval.

5. Concrete and Columns

Concrete is to be measured and delivered to site. It is critical that the concrete ordered, delivered and poured in accordance with the structural specifications. Important considerations include the concrete mix, compaction requirements, delivery waiting time and finish. After the concrete is placed, the installer is to submerge the column in the pier to the required depth ensuring the column is plumb, spacings are correct and the specified backslope is achieved. It is recommended to form a concrete pad mount inside the web at the base of the column to act as a stable support for the sleepers.

6. Sleeper Installation

The sleepers are to be installed by hand where safe to do so or by mechanical means elsewhere. Lifting clamps should be utilised where possible to provide fast and efficient installation while protecting the sleeper from cosmetic damage. Sleepers are to be installed level and in accordance with the structural arrangement. It is critical that care is taken when placing the sleepers to ensure a uniform an aesthetically pleasing finish is achieved.

7. Drainage and Backfill

The installation of the drainage requirements defined by the engineer is critical to prevent excessive pore-water pressure from impacting on the wall. The general methodology and arrangement is as follows:

- Run the specified geosynthetic fabric on the soil side, from the base of the wall to the top of the cut for the entire length of the wall and fix temporarily in place;
- Provide 100mm of drainage material between the wall and the geosynthetic fabric;
- Install 100mm agricultural pipe along the length of the wall and connect or exit the wall at the location specified;
- Continue backfilling the wall with the 20mm drainage gravel stopping 300mm from the top or as described in the construction drawings and fold the geosynthetic fabric over the drainage gravel.

8. Site Demobilisation

On completion of the project the site is to be cleared and left in an appropriate state for further construction activities to proceed if required. The head contractor or client should be notified of the loading limitations of the wall to ensure that these are not exceeding during construction activities. Important client information includes the zone of influence of the wall both in front and behind. Common mistakes include using vibratory rollers above the wall and excavation of service trenches in front of the wall. Bored piers rely on the passive pressure in front of the wall to prevent failure. Excavation in this area can be catastrophic.

3.3.3.3 Cost Estimate

A worked estimate example is provided below for the cantilever bored pier retaining wall. Quantities of materials and labour are a typical representation of a

2m high and 50m long retaining wall. Site access is considered to be adequate with preferable soil conditions. Construction methodologies and plant are considered to be consistent with the construction procedures as previously described. Unit rates for materials and labour have been determined through consultation with various local trades, suppliers and quantity surveyors and represent a fair assessment of the local (SEQ) construction climate at the time of this dissertation. An overall rate of $252.30/m2 has been determined bored pier retaining wall for the height and quantities previously acknowledged.

Activity	Description	Unit	Rate ($)	QTY (Lm)	Total ($)
1	**General**				
1.001	Survey/Set Out	ea	450	0.02	9.00
2	**Materials**				
2.001	Column (150UC23.4) Galvanised 3.5m	Lm	55	1.75	96.25
2.002	Concrete (20MPa pier) 450mm x 2m	m3	165	0.20	33.00
2.003	Sleepers (200x75)	ea	24.5	6.00	147.00
2.005	100mm unsock agg drain	Lm	4.5	1.00	4.50
2.006	Recycled drainage gravel (20mm)	tn	25	0.72	18.00
2.007	A24 Geosynthetic fabric (2000x50m)	Lm	2.54	1.00	2.54
3	**Plant**				
3.001	6.5t Excavator (cut, drill piers and backfill - wet) 4d	day	960	0.08	76.80
4	**Labour**				
4.001	Supervision	day	600	0.01	6.00
4.002	Pier pour and column placement 1d x 2ppl	day	400	0.04	16.00
4.003	Sleeper installation 2.5d x 2ppl	day	400	0.10	40.00
			Sub-Total (Lm)		449.09
			Margin and O/H (10%)		44.91
			Contignecy (3%)		13.47
			Total (Lm)		507.47
			Total m2 (ex GST)		**253.74**

Figure 21: Costing sheet of bored pier retaining wall

3.3.4 Gravity walls

Gravity walls are the earliest known forms of retaining structure and rely solely on their own weight to resist the lateral and hydrostatic forces imposed on them. Due to their size they develop little or no tension and therefore do not require reinforcement. However, as the height of walls increase many gravity walls employ soil reinforcement strategies to improve their performance abilities. Within the gravity category there are a number of different wall types, a selection of these walls are described briefly below.

Crib walls are individual interlocking boxes made from precast reinforced concrete or timber. The boxes are filled with crushed granular material thus providing a free draining structure and therefore not subject to hydrostatic pressures

Figure 22: Crib wall at a height of approximately 4m high (Retaining Solutions 2017)

Segmented block walls are interlocking man made blocks that are easily constructed by hand. In residential construction, segmented block walls are generally used up to 1.2m. However in commercial applications soil reinforcing can be employed to enable segmented walls to achieve much greater heights.

Figure 23: A typical segmented block wall (Dallas Fortworth Retaining Walls 2013)

Gabion walls consist of rectangular steel cages filled with crushed rock. The cages are then stacked to the desired height. In situations where excavation of rock is required, gabion walls utilise the excavated material providing an economic and sustainable wall option.

Figure 24: Large gabion wall in a residential setting (Fine Mesh Metals 2002)

Mass Walls consist of stacked rock such as sandstone blocks as well as formed concrete. Like other gravity walls they rely solely on the weight of materials to resist the lateral earth pressures acting on them. The force diagram below depicts the typical arrangement of forces acting on a gravity walls.

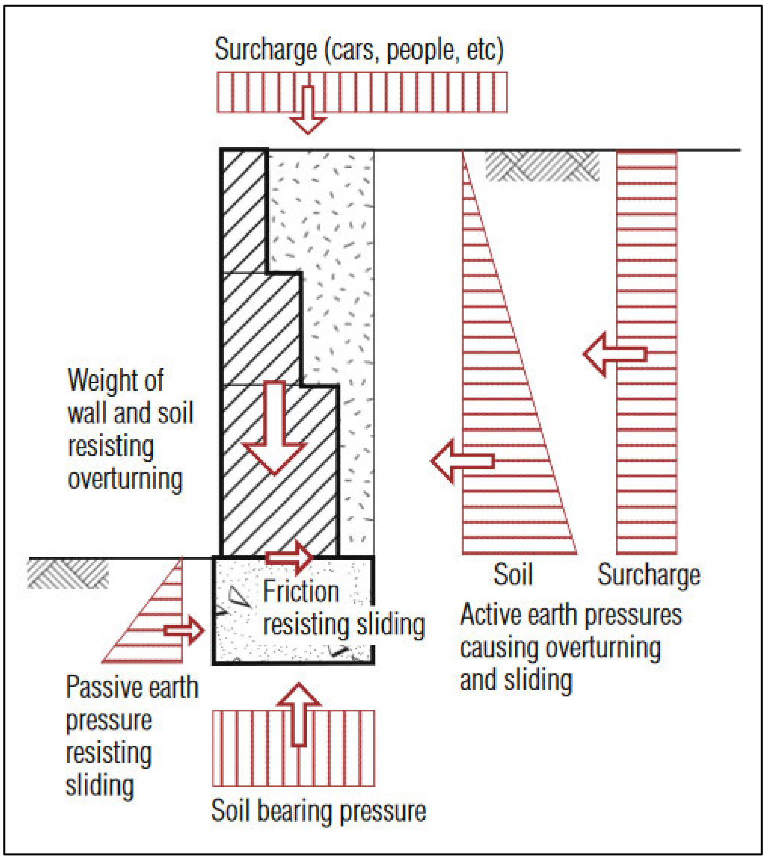

Figure 25: General arrangement of forces acting on a gravity wall. (CCAA 2008)

3.3.4.1 Characteristics

Advantages of gravity walls include:

- Often cheap and utilise simple construction methodologies;
- Aesthetically pleasing where natural minerals such as sandstone are used;
- Potential to recycle and reuse site materials.

The disadvantages associated with gravity walls include:

- Not suitable for soft soils as bearing failure becomes problematic;
- Space is required behind the wall for construction and backfilling;
- Significant depth of wall;

- Restricted to approximately 4.5m high without alternative reinforcement measures.

3.3.4.2 Construction Methodology

The following section sets out the general methodology for the construction of sandstone block gravity retaining walls. The wall is highly common throughout South East Queensland and provides an affordable option particularly when bearing capacities are considered adequate. The methodology covers the trades, materials, plant requirements and general construction procedures. It is assumed that site access and soil conditions are favourable and the wall is being constructed against a cut embankment Therefore, the requirement to import and compact fill in order to achieve site design elevations is not considered. It does however consider the backfilling requirement to satisfy drainage design elements of the wall.

Plant and Material - Sandstone Walls

Trades
Surveyor
Earthworks contractor
Carpenter / Installer / Operator

Materials
Sandstone Bocks 1000 x 500 x 500mm
Foundation aggregate CBR15 or similar
100mm unsocked agricultural pipe
A24 geosynthetic material
Drainage gravel (20mm)
Backfill capping clay

Plant
Excavator (10t)

Concrete Truck

Delivery Vehicles

Construction Procedure

1. Site Establishment

Site establishment is critical to achieving efficient construction practices. Important elements include defining the work zone, locating appropriate lay-down areas and establishing legislative site amenities. Note that in many cases retaining wall contractors are working under minor works subcontracts and amenities are provided by the head contractor in most instances. The contractor is to ensure that all services are clearly marked prior to excavation commencement and that DBYD plans have been collected and assessed. In some instances excavation permits are required to be attained from the head contractor. It is critical where the wall is greater than 1m in height that the retaining structure drawings have been designed and certified by a registered engineer.

2. Survey

A registered surveyor is to be engaged to provide the required mark out for the perimeter of the wall. This generally will include marker stakes indicating the finished face the base of the wall. Note that the civil drawings must be reviewed to ensure the location of the top and/or bottom of the wall coincides with the designed allotment boundaries. The surveyor should provide a site datum to allow the finished reduced levels (top and bottom) to be confirmed.

3. Foundations and Excavations

The contractor is to off-set the survey markers where construction activities may cause them to be lost. The installation of hurdles/profiles can assist to maintain the surveyors mark out and eliminate the requirement of the surveyor to reinstate the points. The foundation excavation is to be in accordance with the dimensions set out in the structural specification. A suitable foundation material is to be compacted in the excavation to provide a stable bearing surface for the sandstone blocks.

WITNESS POINT -The certifying engineer may wish to be present to witness the soil conditions prior to the placement of the foundation base. It is critical to ensure that the natural soil is consistent to the geological survey as the gravity wall relies heavily on the bearing capacity of the soil. Construction should not proceed until the engineer has given approval.

4. Block Installation

The blocks are to be lifted into position using an excavator or light crane. Stringlines should be used to ensure a straight line is achieved where applicable. The required set back of each subsequent level must be maintained in accordance with the structural drawing.

WITNESS POINT - The certifying engineer may wish to attend site to ensure that the completed sandstone block wall is consistent with the design. In particular the engineer will be checking that the set back of subsequent levels has been adhered to and that the sandstone blocks are appropriately positioned on the compacted foundation material.

5. Drainage and Backfill

The installation of the drainage requirements defined by the engineer is critical to prevent excessive pore-water pressure from impacting on the wall. The general methodology and arrangement is as follows:

- Run the specified geosynthetic fabric on the soil side, from the base of the wall to the top of the cut for the entire length of the wall and fix temporarily in place;
- Provide 100mm of drainage material between the wall and the geosynthetic fabric;
- Install 100mm agricultural pipe along the length of the wall and connect or exit the wall at the location specified;

- Continue backfilling the wall with the 20mm drainage gravel stopping 300mm from the top or as described in the construction drawings and fold the geosynthetic fabric over the drainage gravel;
- Compact a clay soil in the form of a water resistant plug to complete the backfill. The clay layer acts to prevent the ingress of surface water behind the wall which may contribute to pore-water pressures.

6. Site Demobilisation

On completion of the project the site is to be cleared and left in an appropriate state for further construction activities to proceed if required. The head contractor or client should be notified of the loading limitations of the wall to ensure that these are not exceeding during construction activities. Important client information includes the zone of influence of the wall both in front and behind. Common mistakes include using vibratory rollers above the wall and excavation of service trenches in front of the wall.

3.3.4.3 Cost Estimate

A worked estimate example is provided below for the sandstone block gravity retaining wall. Quantities of materials and labour are a typical representation of a 2m high and 50m long retaining wall. Site access is considered to be adequate with preferable soil conditions. Construction methodologies and plant are considered to be consistent with the construction procedures as previously described. Unit rates for materials and labour have been determined through consultation with various local trades, suppliers and quantity surveyors and represent a fair assessment of the local (SEQ) construction climate at the time of this dissertation. An overall rate of $206.52/m2 has been determined for the sandstone retaining wall for the height and quantities previously acknowledged.

1	**General**				
1.001	Survey/Set Out	ea	450	0.02	9.00
2	**Materials**				
2.001	Compacted foundation aggregate CBR15 or similar (1000x450mm)	tn	28	0.68	18.90
2.002	Sandstone Blacks (500x500x1000mm)	ea	35	4.00	140.00
2.005	100mm unsock agg drain	Lm	4.5	1.00	4.50
2.006	Recycled drainage gravel (20mm)	tn	25	0.72	18.00
2.007	A24 Geosynthetic fabric (2000x50m)	Lm	2.54	1.00	2.54
3	**Plant**				
3.001	10t Excavator (excavte/compact foiundation, sling blocks, backfill - wet hire) 6d	day	1120	0.12	134.40
4	**Labour**				
4.001	Supervision	day	600	0.01	6.00
4.002	Block Installation 4d x 1 pp	day	400	0.08	32.00
			Sub-Total (Lm)		365.34
			Margin and O/H (1(		36.53
			Contignecy (3%)		10.96
			Total (Lm)		412.83
			Total m2 (ex GST)		**206.42**

Figure 26: Costing sheet of sandstone gravity wall

3.4 Bored Pier Retaining Wall Controversy

A recent publicised event relating to bored pier retaining wall failures in South East Queensland promoted the author to further investigate the design and construction of this extensively used retaining wall. Figure 27 displays the catastrophic failure of the inadequately designed wall in Gold Coast, QLD.

Figure 27: Gold Coast wall failure (Brisbane Times 2017)

Theoretical design methodologies for the bored pier wall are available, however it was concluded that the designs are generally an adaptation of other retaining structures such as that of the sheet pile wall. Fundamental design methodologies for cantilever walls on spread foundation or gravity walls are common place in structural and geotechnical texts. The justification for further analysis of the design methods pertaining to the bored pier retaining wall are established on the basis of the need for uniformity to provide a common understanding of design requirements.

Investigations into the practice of bored pier retaining wall installation across South East Queensland including discussions with consulting engineers and civil contractors. The resultant findings of the investigations are summarised below:

- Bored pier retaining wall installation contracts are generally only considered in a design and construction capacity. This is contrast to that of other walls with established design methodologies such as masonry block cantilever walls. It is the opinion of the author that a lack of industry confidence in the performance of these walls is evident.
- Supervision by suitably qualified personnel across the board is generally poor. Figure 28 shows a wall recently constructed in Brisbane, QLD to a height of 3.5m. Of immediate concern was the wall had been cut against the foundations of a residential structure without the use of a shoring support mechanism such as sheet piling. The result was displacement of the slab foundation and excessive damage to the structure. It is also evident that insufficient space is available behind the columns to provide the necessary drainage materials. This is a clear indication that a lack of construction and engineering knowledge by site supervisors was apparent at this land development site.
- Quality of installation is considered generally poor. Figure 29 displays a recently completed wall in Gold Coast, QLD. Poor quality and workmanship is evidenced by the undulating sleeper alignment. A lack of suitably skilled subcontractors can be evidenced in this example.

The bored pier retaining wall has been justifiably selected to undergo further analysis based on a gap in theoretical design knowledge and the results of the industry consultation as described above.

Figure 28: Brisbane wall displaying inadequate construction and engineering knowledge

Figure 29: Gold Coast wall evidencing poor workmanship

Chapter Four: Methodology

4.1 Introduction

The proposed methodologies for this project are primarily based on the research and investigation findings presented in the literature review.

The analysis aims to determine inefficiencies and non-compliance elements of design alternatives in accordance with relevant Australian Standards. The structural analysis will be undertaken using theoretical calculations and verified using finite element software, Strand7.

4.2 Design Condition

The following design condition has been selected to provide a realistic scenario for which to base the design analysis. A cohesionless soil condition has been selected to provide a uniform triangular active pressure (Pa) and is commonly found in coastal regions such as South East Queensland. Due to the unpredictability and uncertainty of soil conditions experienced across a site, it is considered reasonable to assume that the soil may act in a cohesionless manner despite the result of localised bore hole samples. Designers should consider the suitability of geotechnical surveys across the whole of the retaining wall area. A simplistic loading condition of horizontal backfill with a 5kPa surcharge (Standards Australia 2002) has been provided for the purpose of clarity.

4.2.1 Soil Parameters

Table 1: Effective Soil Parameters

Soil Parameters	
Unit weight, γ' (kN/m3)	18
Internal angle of friction, ϕ' (deg)	35
Cohesion, c' (kPa)	0

Partial factors for cohesion and the internal angle of friction in accordance with AS 4678 are taken below.

ϕ'_{uc} = 0.75;

c^* = 0.75 c' (5.5.1)

$\phi'_{u\Phi}$ = 0.9;

ϕ'^* = $\tan^{-1}(\phi_{u\phi}(\tan\phi))$; (5.5.2)

By applying the equations above the internal angle of friction, (ϕ'), becomes 32.2°. Note that effective parameters must be used in the evaluation of soils for retaining walls in accordance with cl. 3.2.1 of AS4678.

4.2.2 Loading Calculations

The wall will experience lateral pressures from both the soil and the surcharge loading. The literature review identified Rankine's theory of lateral earth pressure as the most reliable and predictable method for use under these conditions. As described in figure 30 below, the lateral earth pressure is a triangular distribution that increases with depth while the surcharge loading is a uniformly distributed load.

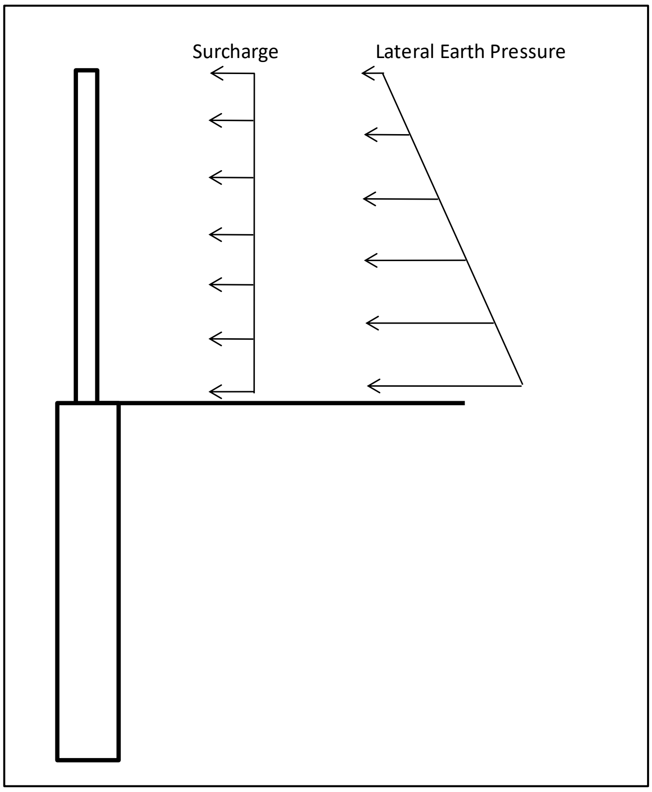

Figure 30: Loading arrangement of design condition

Equation 26 is used to determine the active pressure exerted by the loading condition on the sleeper and column elements.

$$\sigma'_a = Ka * \gamma * H + Ka * q \tag{26}$$

Standards Australia (2002) states the loading is to be factored as 1.25G + 1.5Q and describes both lateral earth pressure and imposed earth pressure (surcharge loading) as dead loads. Live loads such as earthquake, wind and hydrostatic loadings are not considered in this design.

4.3 Bored Pier Retaining Wall Overview

The bored pier retaining wall was selected to undergo further analysis due to its multiple design iterations, extensive use in the civil construction industry, public failures and controversy as identified in chapter 3.

Figure 31: 4.5m high wall Springfield, QLD (Tan, C 2016)

The wall is well defined in its componentry allowing an accurate analysis to be undertaken on each element using a predefined design loading condition. Each member and analysis methodology is discussed in further detail.

The wall can be commonly found throughout residential land developments at heights between 0.2m and 3.0m and in rare instances up to 4.5m. For the purpose of this dissertation we will consider a design height of between 0.2m and 3.0m. For the purpose of this analysis, passive pressures in front of the wall are considered to be present at the base of the wall. It is noted that cl. 4.4.7 of AS2159 details the requirements of the engineer to determine an appropriate depth to which the soil in front of a pile has been compromised due to water ingress and disturbance. This soil is to be discounted from passive resistance calculations.

Figure 32: 4.5m high wall in Springfield, QLD (Roberts, D 2016)

4.4 Sleeper Analysis

4.4.1 Introduction

The sleepers or beams used in the retaining system are a reinforced concrete element which must withstand the lateral earth pressures and surcharge loads imposed on the wall. The compressive load is transferred along the length of the sleeper and onto the posts and hence the analysis is focused on the sleeper's flexural and shear strength capacity. It is noted that serviceability of the sleeper is not considered in this study on the basis that deflection of the beam will not affect the overall performance of the wall and impacts to the aesthetics of the wall are likely to result from failure of the sleeper in flexural strength rather than deflection.

The sleepers are constructed locally at precast yards throughout South East Queensland and available for purchase direct from suppliers or alternatively through various construction/hardware retailers. Parameters used in this analysis are replications of standard sleepers manufactured and used in the South East Queensland civil construction industry. Note that many of the sleeper designs are made publicly available on the manufacturer's websites.

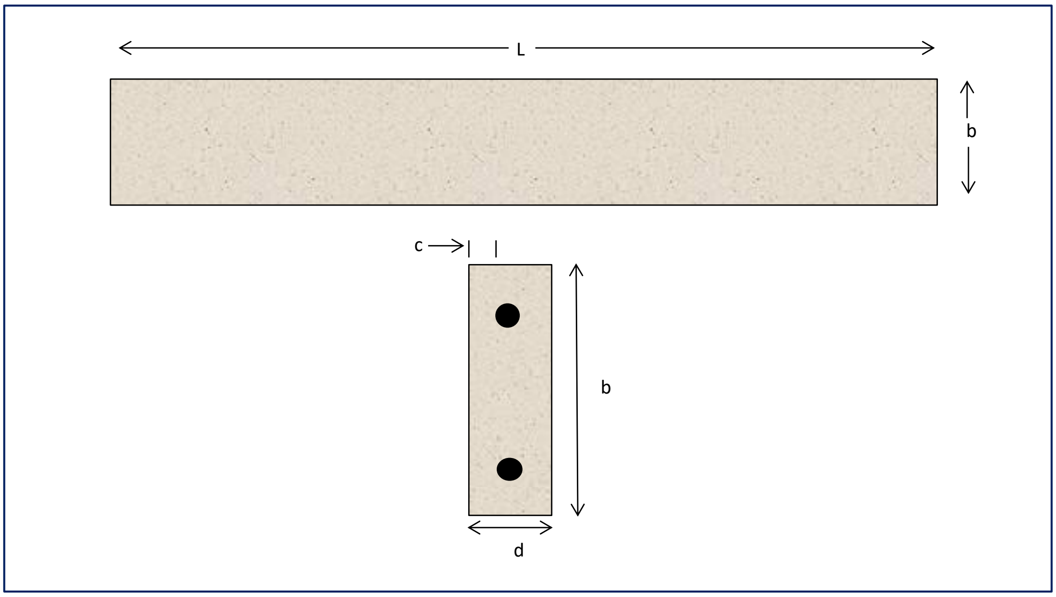

Figure 33: Geometric arrangement of sleeper

4.4.2 Calculations

The sleepers are to be analysed for flexural and shear strength when subjected to the distributed loads applied by the lateral earth and surcharge loadings. The bending moment and shear force are to be calculated from theoretical analysis in accordance with AS3600 - Concrete Structures. The universally distributed load for each sleeper is to be derived from the proposed loading condition.

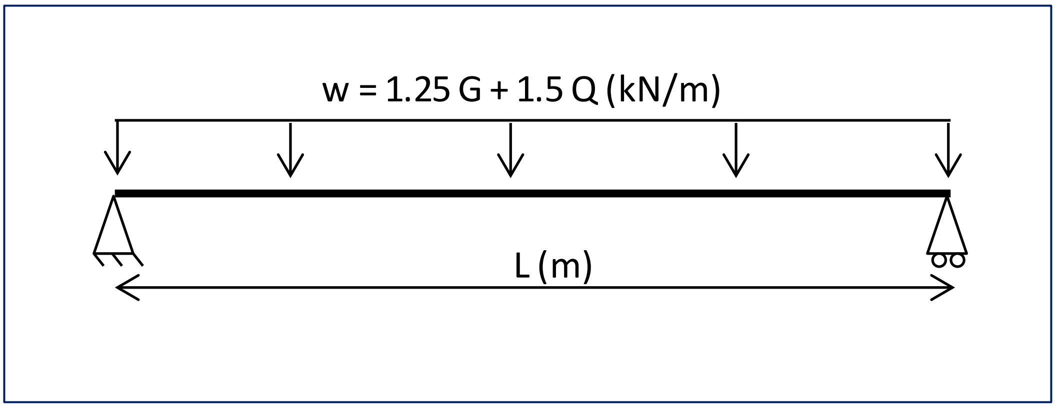

Figure 34: Line load diagram for sleeper

Bending Moment (M* and ϕM)

The following equations are to be used to determine the bending moment experienced by the sleepers under load (M*) and the flexural strength capacities of the design alternatives (ϕM) in accordance with AS 3600 - Concrete Structures.

M*

The equation for the experience bending moment of the member is:

$$M = \frac{wl^2}{8} \tag{27}$$

ϕM

Section 8.1 of AS3600 defines the requirement for the evaluation of the strength of beams in bending. Figure 35 presents the arrangement for the determination of stresses within the element cross section.

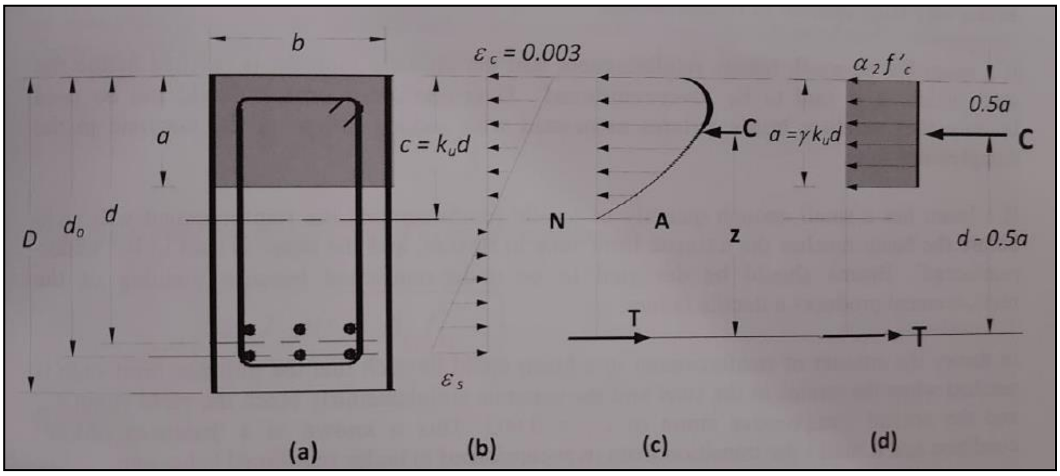

Figure 35: Stress arrangement in singularly reinforced concrete beams (Beletich et. al 2013)

where;

$\alpha_2 = \ 1.0 - 0.003f'c \quad (0.67 \leq \alpha_2 \leq 0.85)$

$\gamma = 1.05 - 0.007f'c \quad (0.67 \leq \gamma \leq 0.85)$

$$Ku = \frac{1}{\alpha_2\gamma} + \frac{A_{st}}{bd} + \frac{f_{sy}}{f'c} \tag{28}$$

The equation for the moment capacity:

$$\phi M_{uo} = \phi A_{st} f_{sy} d \left[1 - \frac{1}{2\alpha_2}\frac{A}{bd}\frac{f_{sy}}{f'c}\right] \tag{29}$$

where;

$\phi = \ 0.6 \leq (1.19 - 13kuo/12) \leq 0.8$ (Table 2.2.2)

Shear Force (V* and ϕV)

The following equations are to be used to determine the shear force experienced by the sleepers under loading (V*) and the shear strength capacities of the design alternatives (ϕV) in accordance with AS 3600 - Concrete Structures.

V*

The equation for the experienced shear force in the member is:

$$V = \frac{wl}{2} \tag{30}$$

ϕV

Section 8.2.7.1 of AS3600 defines the requirement for the evaluation of beam with no shear reinforcement. The shear capacity of the design alternatives can be defined using the following equation:

$$\phi Vuc = \beta 1\, \beta 2\, \beta 3\, bv\, do\, f'cv \left[\frac{Ast}{bv\, do}\right]^{1/3} \qquad (31)$$

where:

$\phi = 0.7$ (Table 2.2.2)

$\beta 1 = 1.1(1.6 - do/10000) \geq 1.1$

$\beta 2 = 1$ *(members subject to pure bending)*

$\beta 3 = 1$

$f'cv = f'c^{\frac{1}{3}} \leq 4MPa$

4.5 Column Analysis

4.5.1 Introduction

The columns used in the retaining system are a steel column or universal beam which must house the sleepers in the internal web with the flange acting as the bearing support. The post must provide sufficient support of the imposed load from the sleepers and transfer it to the pile at the base of the post. The embedment depth of the post is assumed to be full depth of the pier with 100mm cover at the base. Analysed as cantilever beam, the post is subject to strength and serviceability analysis. A serviceability limit of 1:100 of horizontal displacement at the free head of the column is considered acceptable in accordance with AS 4678. The analysis is to be conducted in accordance with AS4100 - Steel Structures with all sectional data to be collated using the OneSteel properties catalogue (OneSteel n.d.). The columns are hot dip galvanised to provide sufficient corrosion protection in accordance with AS/NZS 2312: 2002 Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings.

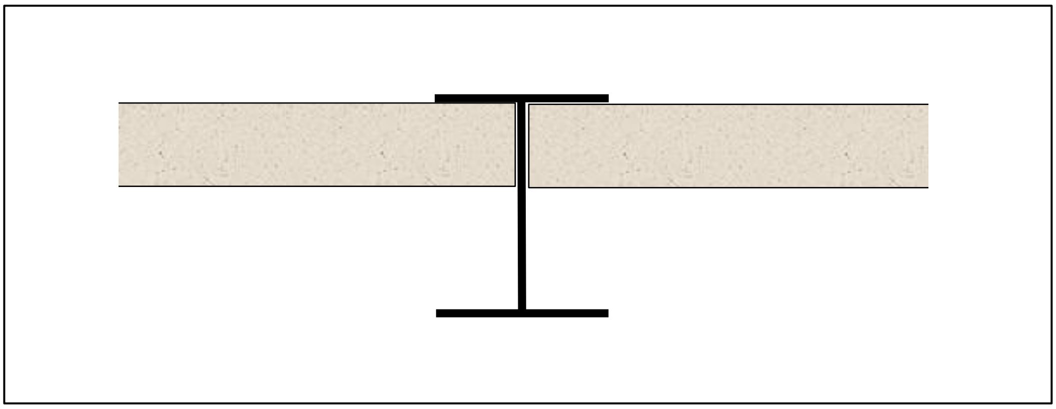

Figure 36: Sleeper housing in web of column

4.5.2 Calculations

Bending Moment (M* and ϕM)

The following equations are to be used to determine the bending moment experienced by the columns under load (M*) and the flexural strength capacities of selected universal columns or beams (ϕM) in accordance with AS 4100 - Steel Structures.

M*

The column will be subjected to the loading equal to the column spacing width. The column is subjected to both lateral earth pressure due to soil parameters as well as the defined surcharge loadings. Equation 31 has been derived to provide the maximum moment experienced by the column.

$$M^* = \left(\frac{w_G * h}{l} * \frac{h}{3}\right) + \left(w_q * h * \frac{h}{2}\right) \tag{31}$$

ϕM

The nominal section capacity of the columns is calculated in accordance with AS4100- Steel Structures. The reinforced concrete sleepers provide full lateral restraint to the column lending the nominal capacity to be calculated by equation 32.

$$\phi M = \phi fsy\, Zex \tag{32}$$

where:

ϕ = *0.9* (Table 2.2.2)

Shear Force (V* and ϕV)

V*

The shear force experienced is subject to the lateral earth pressure and surcharge loading applied to the retaining wall. The following equation has been derived to determine the maximum shear stress experienced by the column member.

$$V = \left(\frac{w_G * h}{l}\right) + \left(w_q * h\right) \qquad (33)$$

ϕV

When subjected to the design loadings the column will develop compression and tension shear stress components. As such, the yield stress will govern the capacity of the member. The yield capacity can be calculated by the following equation.

$$\phi V = 0.6\, fy\, Aw \qquad (34)$$

4.5.2.1 Deflection (Serviceability)

A deflection limit of 1:100 at the free head of the column provides an achievable standard whilst maintaining the aesthetic integrity of the completed structure. It should be noted that the lateral displacement at the free head of the pile will have a greater effect on the overall horizontal displacement at the top of the wall. The columns are subjected to a uniformly distributed surcharge load as well as the triangular distribution load of the lateral earth pressure. The deflection equations for a cantilever beam subjected to the aforementioned loading arrangement are combined to give the following for the maximum deflection experienced by the column.

$$\delta_{max} = \frac{w_S L^4}{8EI_{xx}} + \frac{w_G L^4}{30EI_{xx}} \qquad (35)$$

4.6 Pier

4.6.1 Introduction

The pier can be considered the most critical aspect of the retaining wall design due to its dependence on the localised soil conditions. The pier consists of an auger excavated cylindrical foundation, mass filled with concrete to a depth and diameter determined by the design engineer. The pier supports the deadweight of the wall along with the moment due to passive and active earth pressure.

A variety of column embedment arrangements are currently used in industry practise, however for the purpose of this project we will utilise a commonly used method where the column is cast monolithically with the pier to a depth 100mm above the bottom the foundation.

It is important to note that geotechnical surveys are generally spaced in a grid formation and in some instances over 50m apart. It is highly unlikely that the soil parameters at each pier location can be accurately determined. It stands to reason that a conservative factor of safety should be applied to the passive resistance of the soil in order to account for the potentially unknown strata parameters. For this project a passive pressure factor of safety of 1.5 is to be used.

4.6.2 Calculations

4.6.2.1 Soil Arching

For the determination of the spacing and depth required by the pier, the wall is to be analysed as a solid pile wall similar to that of a sheet pile wall.

To do this a soil arching factor is to be determined in order to account for the spacing between the piles and subsequently the additional loadings the piles are subjected to.

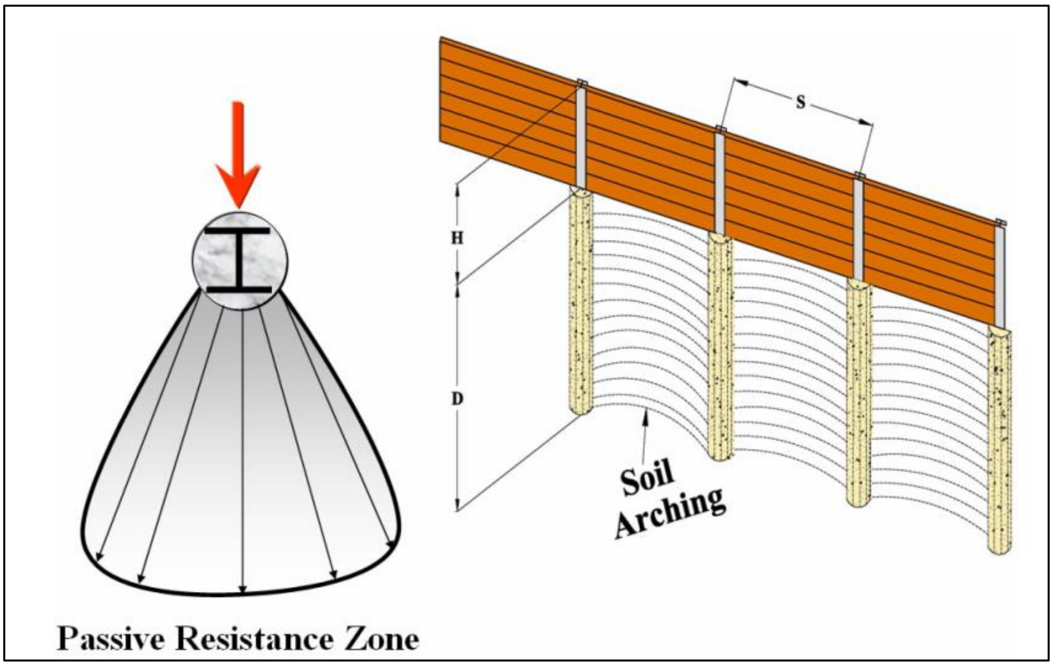

Figure 37: Soil arching and passive resistance zone (Department of Transportation 2011)

The arching factor is determined by multiplying an adjustment factor by the effective pile width then dividing by the spacing as equation 36 below. The factor is then applied to all equations of earth pressures below the excavation line.

$$f = \frac{0.08\phi * d}{S} \tag{36}$$

where

0.08ϕ = adjustment factor

d = effective pile width

S = pile spacing

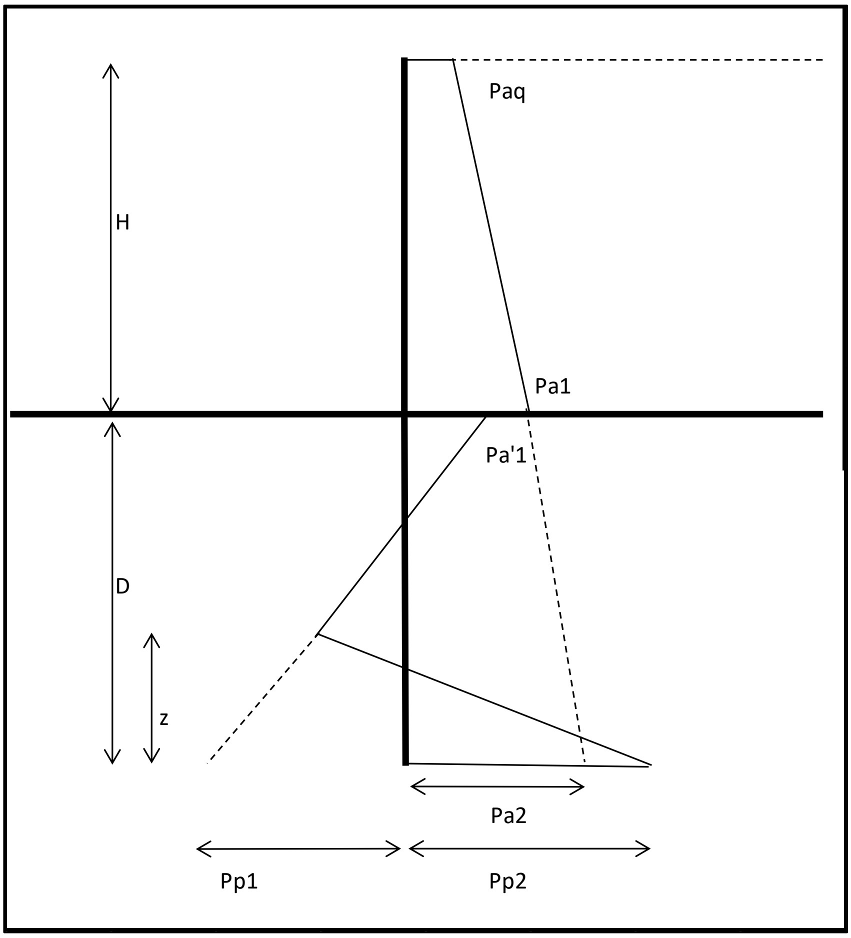

Figure 38: Force diagram of bored pier retaining wall

4.6.2.2 Pier Depth

Use Rankine theory to determine the soil coefficients:

$$\mathrm{Ka} = \frac{1-\sin(\phi)}{1+\sin(\phi)}$$

$$\mathrm{Kp} = \frac{1+\sin(\phi)}{1-\sin(\phi)} * (FOS)$$

Compute the active and passive pressures:

$$Paq = Ka * q$$

$$Pa1 = Ka * \gamma * H$$

$$Pa'1 = f * Kp * \gamma * H$$

$$Pa2 = f * Kp * \gamma * H + Pa'1$$

$$Pp1 = f * \gamma * D * (Kp - Ka) - Pa'1$$

$$Pp2 = f * \gamma * D * (Kp - Ka) + f * Kp * \gamma * H$$

Calculate the depth of rotation (z):

$$z = ((Pp1 - Pa'1) * D - H * Pa1 - H * Paq)/(Pp1 + Pp2) \quad (37)$$

Calculate the forces:

$$F1 = Paq * H$$

$$F'1 = f * Ka * q$$

$$F2 = \frac{1}{2} * Pa1 * H$$

$$F3 = (Pa'1 + Pa2) * \frac{D}{2}$$

$$F4 = (Pp1 + Pp2) * \frac{z}{2}$$

$$F5 = (Pp1 + Pa2) * \frac{D}{2}$$

Calculate the sum of the moments about the base of the pile:

$$\Sigma M = F1 * \left(\frac{H}{2} + D\right) + F'1 * \frac{D}{2} + F2 * \left(\frac{H}{3} + D\right) + Pa'1 * \frac{D^2}{2} - (Pa2 - Pa'1) * \frac{D}{2} * \frac{D}{3} + F4 * \frac{z}{3} - F5 * \frac{D}{3} \quad (38)$$

Solving the equation for ΣM = 0 gives the required depth of the pier to resist overturning.

Find Point of Zero Shear (Maximum Moment):

Zero shear will occur at point B. First we need to calculate the distance to point A and the corresponding shear force.

$$V_A = \frac{1}{2} * Pa1 * H + Pa'1 * \frac{Y}{2} + Paq * H + f * Paq * Y \tag{39}$$

where

$$Y = \frac{Pa'1}{(f\gamma(Kp - Ka)}$$

V_B is equal to the net passive pressures minus the surcharge loading

$$V_B = f * \gamma * (Kp - Ka) * X * \frac{X}{2} - f * Paq * X \tag{40}$$

We can now determine X as the point where the passive pressure equals the active pressure. Therefore V_B - V_A = 0.

$$f * \gamma * (Kp - Ka) * X * \frac{X}{2} - f * Paq * X - \frac{1}{2} * Pa1 * H - Pa'1 * \frac{Y}{2} - Paq *$$

$$H + f * Paq * Y = 0 \tag{41}$$

Solving equation 41 for X gives the point of maximum bending moment.

Calculate the maximum bending moment (B):

The maximum bending moment experience by the pier at X is determined by equation 42.

$$\Sigma M_B = Pa1 * \frac{H}{2} * \left(\frac{H}{3} + Y + X\right) + Pa'1 * \frac{Y}{2} * \left(X + \frac{2}{3Y}\right) - f * \gamma * (Kp - Ka) *$$

$$X * \frac{X}{2} + Paq * H * \left(\frac{H}{2} + D\right) + f * Paq * D * \frac{D}{2} \tag{42}$$

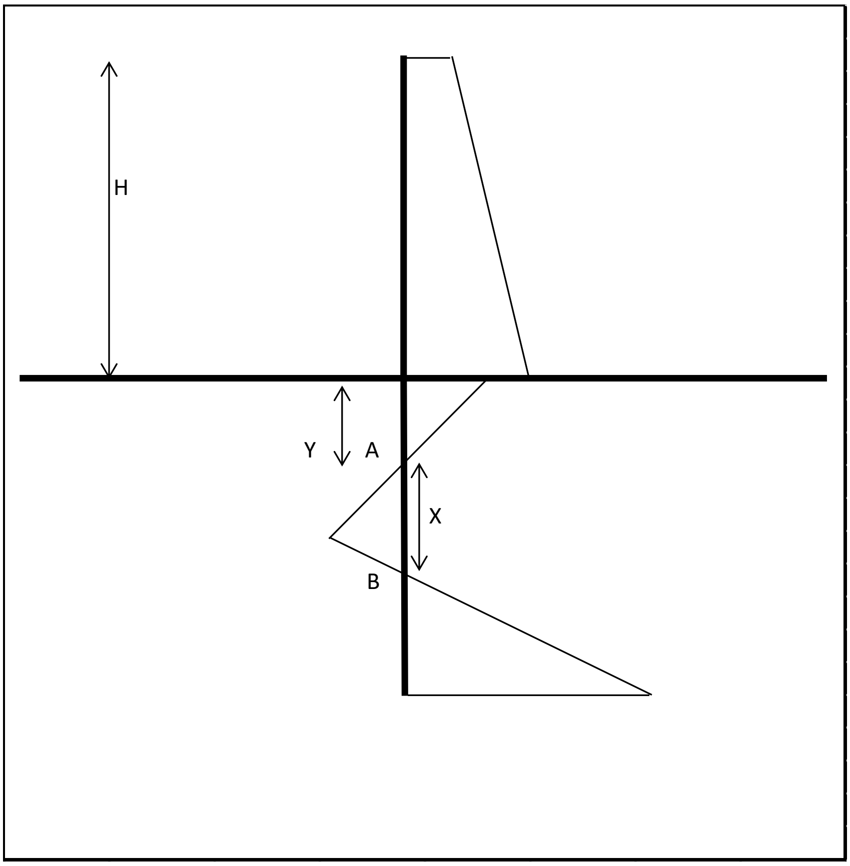

Figure 39: Force diagram of bored pier retaining wall (2)

Chapter Five: Results

5.1 Introduction

This chapter presents the results of the loading calculations based on the predefined design condition and compares them to the capacity of the design alternative's member elements. Both the design condition and member capacities have been calculated in accordance with the following design standards:

- AS3600 - Concrete Structures
- AS4100 - Steel Structures
- AS2159 - Piling- Design and Installation
- AS4678 - Earth Retaining Structures

The methodologies associated with the results have been identified in chapter 4 and strictly adhered to throughout the analysis process.

5.2 Design Condition

5.2.1 Rankine Pressure Coefficients

Ka = 0.305

Kp = 3.282

5.2.2 Lateral Pressure Results

The lateral earth pressure, (σ'_a), acting on the sleeper and column elements has been defined as the sum of the triangular distribution of the active earth pressure and the uniform distribution of the 5kPa surcharge load. Below is an example of the calculation at a depth of 2m including load factoring of 1.25G + 1.5Q.

$$\sigma'_a = 1.25 * Ka * (q + \gamma * H)$$

$$\sigma'_a = 1.25 * 0.305 * (5 + 18 * 2)$$

$$\sigma'_a = 15.618 \text{ kN/m}^2$$

The table below displays the resulting factored active earth pressures at depth increments of 200mm.

Table 2: Resulting lateral earth pressure from design condition loading

Height	Load (KN)	Factored Load (kN/m2)
0.2	2.621	3.276
0.4	3.718	4.647
0.6	4.815	6.019
0.8	5.912	7.390
1.0	7.009	8.761
1.2	8.106	10.132
1.4	9.203	11.504
1.6	10.300	12.875
1.8	11.397	14.246
2.0	12.494	15.618
2.2	13.591	16.989
2.4	14.688	18.360
2.6	15.785	19.732
2.8	16.882	21.103
3.0	17.979	22.474

5.3 Sleeper Analysis

5.3.1 Sleeper Parameters

The sleepers noted below represent commonly available sleepers manufactured in South East Queensland. The analysis has been conducted in accordance with AS3600 - Concrete Structures and consistent with the methodology set out in Chapter 4.

Table 3: RCC sleepers to undergo analysis

Reinforced Concrete Cement Sleepers								
#	**L(mm)**	**b(mm)**	**d(mm)**	**f'c**	**No. bars**	**dia. (mm)**	**fsy (Mpa)**	**cover(mm)**
1	1000	200	75	40	2	12	500	30
2	1500	200	75	40	2	12	500	30
3	2000	200	75	40	2	12	500	30
4	2000	200	75	40	3	12	500	30
5	2000	200	80	40	3	12	500	30
6	2000	150	75	50	2	10	500	25
7	2000	200	100	40	2	16	500	30
8	3000	200	100	40	3	16	500	30

5.3.2 Theoretical Loading Results

The maximum bending moment and shear force exerted on the sleeper has been calculated for the various heights of the wall and tabulated below.

Table 4: Theoretical maximum bending moment and shear force on sleepers

Design Condition Bending and Shear Force						
Height	**2m Spacing**		**1.5m Spacing**		**1.0m Spacing**	
	M*	**V***	**M***	**V***	**M***	**V***
0.2	1.639	3.279	0.922	2.459	0.410	1.639
0.4	2.326	4.651	1.308	3.488	0.581	2.326
0.6	3.012	6.024	1.694	4.518	0.753	3.012
0.8	3.698	7.396	2.080	5.547	0.925	3.698
1.0	4.384	8.769	2.466	6.577	1.096	4.384
1.2	5.071	10.141	2.852	7.606	1.268	5.071
1.4	5.757	11.514	3.238	8.635	1.439	5.757
1.6	6.443	12.886	3.624	9.665	1.611	6.443
1.8	7.129	14.259	4.010	10.694	1.782	7.129
2.0	7.816	15.631	4.396	11.723	1.954	7.816
2.2	8.502	17.004	4.782	12.753	2.125	8.502
2.4	9.188	18.376	5.168	13.782	2.297	9.188
2.6	9.874	19.749	5.554	14.812	2.469	9.874
2.8	10.557	21.121	5.945	15.843	2.647	10.568
3.0	11.247	22.494	5.940	15.841	2.640	10.561

5.3.3 Finite Element Analysis Loading Results

Stand7 finite element software has been used to confirm the resulting maximum bending moment and shear force experienced by the sleeper. The sleeper lengths of 1.0m, 1.5m and 2m were analysed at depths of 1m, 2m and 3m and the resulting moment and force were compared with the theoretical results in table 4. The finite element analysis confirms that the theoretical analysis was accurate and suitable for practical use.

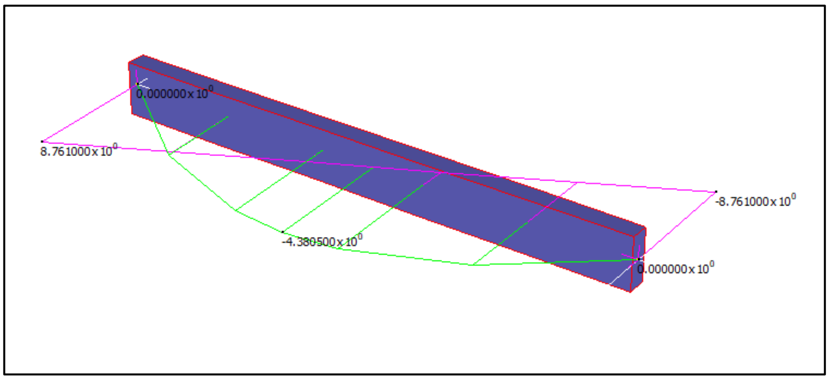

Figure 40: 2m Spacing (1m depth)

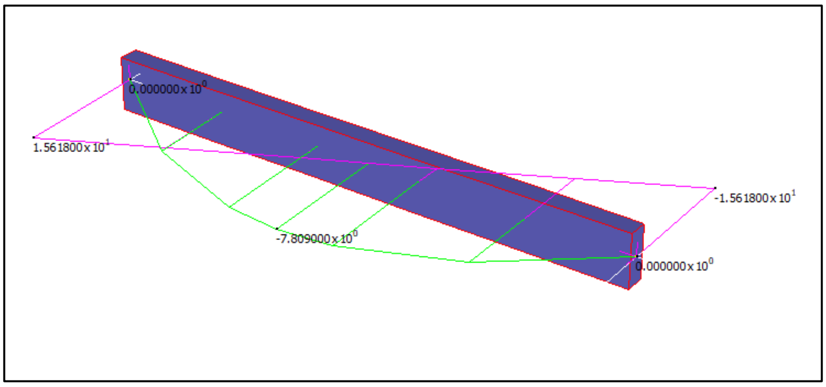

Figure 41: 2m Spacing (2m depth)

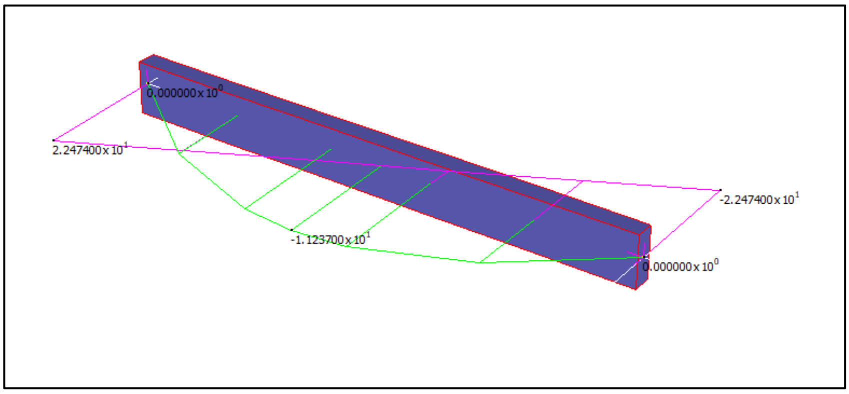

Figure 42: 2m Spacing (3m depth)

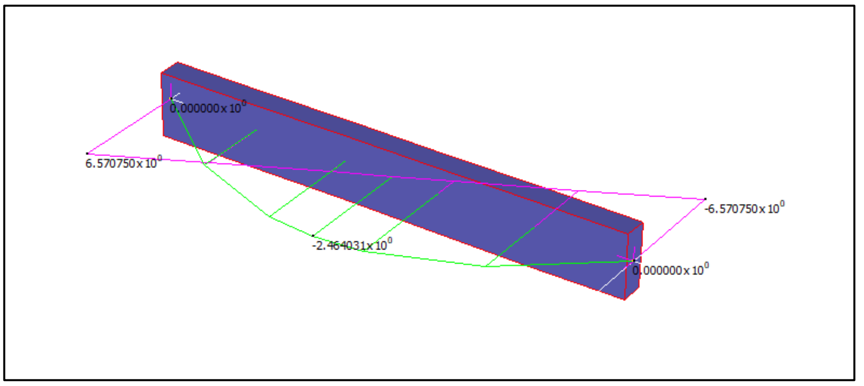

Figure 43: 1.5m Spacing (1m depth)

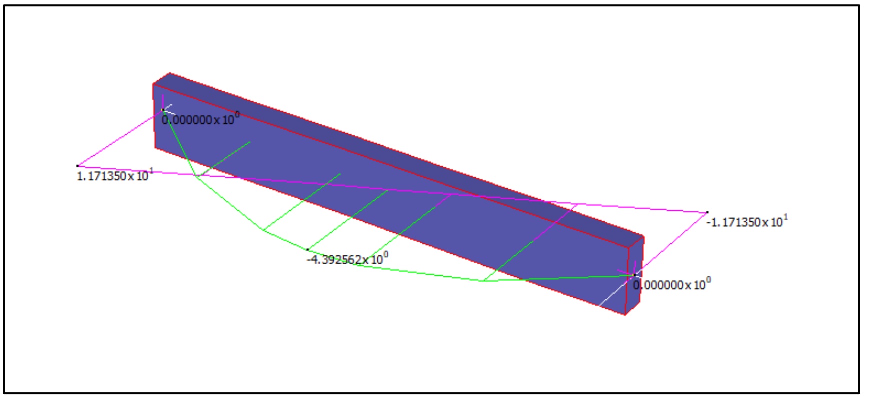

Figure 44: 1.5m Spacing (2m depth)

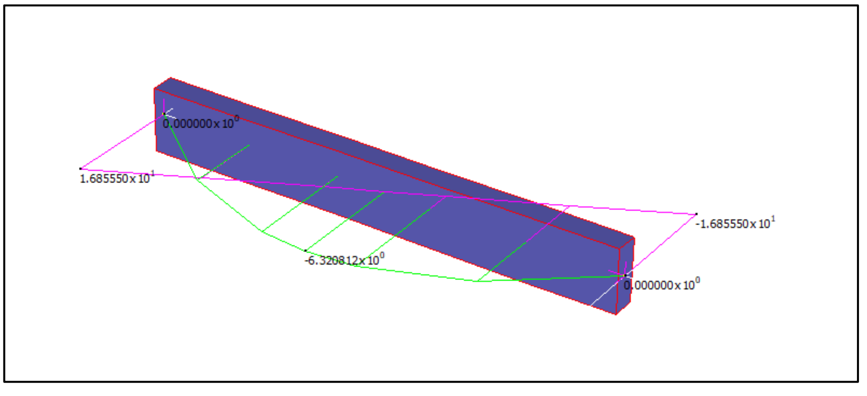

Figure 45: 1.5m Spacing (3m depth)

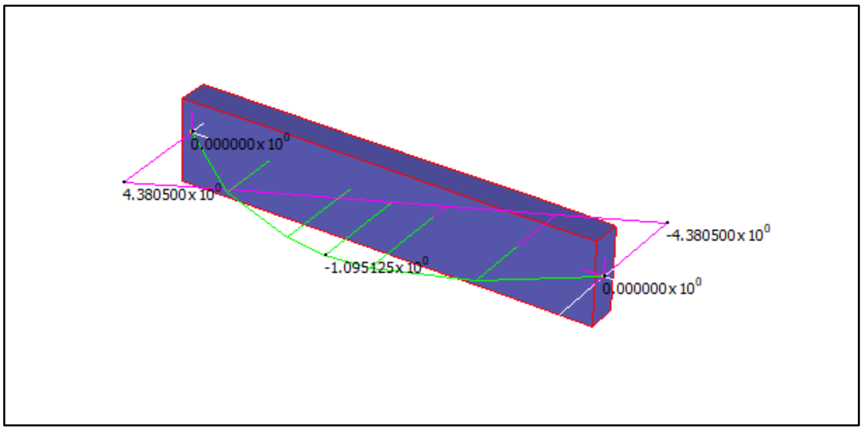

Figure 46: 1.0m Spacing (1.0m depth)

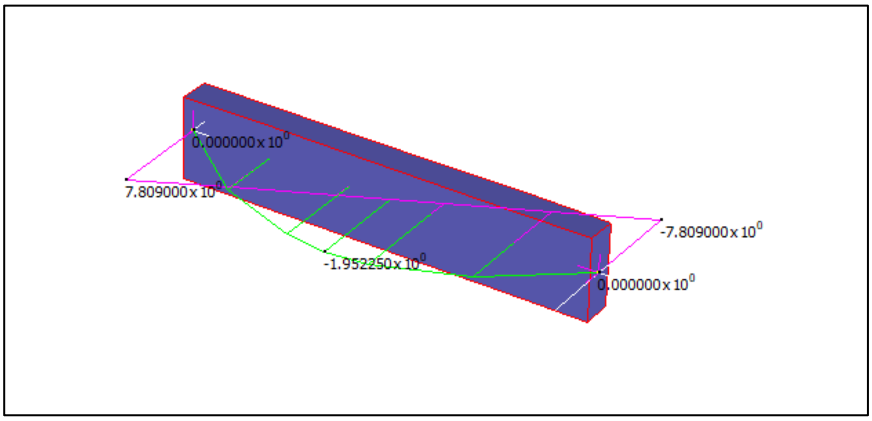

Figure 47: 1.0m Spacing (2m depth)

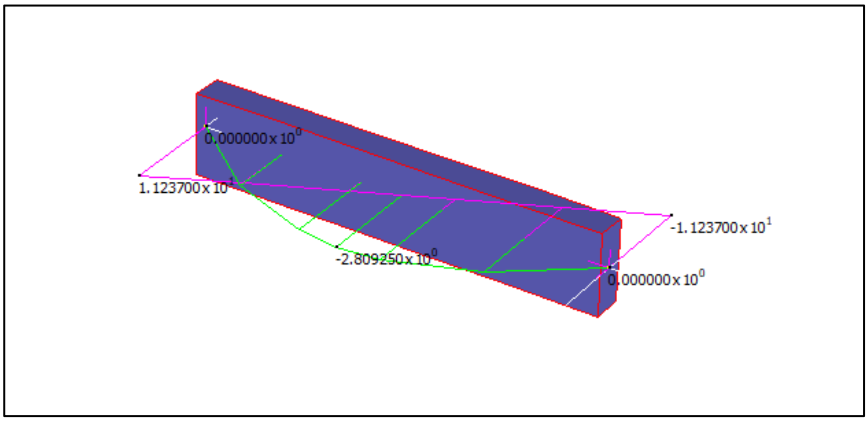

Figure 48: 1.0m Spacing (3m depth)

5.3.4 Sleeper Capacity (Bending and Shear)

The capacity of the 8 presented sleepers have been analysed and calculations shown below. Due to the equivalent geometry and reinforcement detail, sleepers 1, 2 and 3 will share the same strength capacities.

Table 5: Theoretical sleeper capacities

Reinforced Concrete Cement Sleepers Capacities		
Type	**ΦM (kNm)**	**ΦV (kN)**
1,2,3	2.082	9.581
4	2.700	11.276
5	3.209	12.181
6	2.388	8.501
7	5.696	16.051
8	7.206	18.333

Sleepers 1, 2 and 3 Calculations

Design Flexural Strength

d_o $= 75 - 30 - 12 / 2$

$= 39\text{mm}$

Ast $= 2 * (\pi * 12^2) / 4$

$= 226.2\text{mm}^2$

γ_{min} $= 1.05 - 0.007 * 40$ (8.3.1(1))

$= 0.77$

α_2 $= 1 - 0.003 * 40$ (8.1.3(2))

$= 0.85$

ku $= \frac{1}{\alpha 2\gamma} + \frac{Ast}{bd} + \frac{fsy}{f}$

$= 1 / (0.85 * 0.77) + 226.2 / (200*75) + 500 / 40$

$= 0.5538 > 0.36$ **steel will not yield**

Compression controlled

c $= \text{ku d}$

= 0.5538 *39

= 21.56 mm (Neutral Axis)

a = γ ku d

= 0.77 * 21.56

= 16.63

M* = $0.85 * f'c * b * a * \left(d - \frac{a}{2}\right)$

= 0.85 * 40 * 200 * 16.63 * (39 - 16.63 / 2)

= 3.4703 kNm

ϕ = 0.6 ≤ (1.19 – 13ku / 12) ≤ 0.8 (Table 2.2.2)

= 0.59 ≤ 0.6 adopt 0.6

ϕM* = 3.4703 * 0.6

= 2.082 kNm

<u>Design Shear Strength</u>

Vuc = $\beta 1\, \beta 2\, \beta 3\, bv\, do\, f'cv\, [\frac{Ast}{bv\, do}]^{1/3}$ (8.2.7.1)

β_1 = 1.1* (1.6 - d_o / 1000) ≥ 1.1

= 1.71

β_2 = 1

β_3 = 1

f'_{cv} = $f'c^{1/3}$ ≤ 4Mpa (8.2.7.1)

= 3.42 Mpa

V* = 1.71 * 1 * 1 * 200 * 39 * 3.42 $(226.2 / (200 * 39))^{1/3}$

= 14.073 kN

ϕ = 0.7 (Table 2.2.2)

ϕV* = 0.7 * 14.073

= 9.851 kN

Sleeper 4 Calculations

Design Flexural Strength

d_o = 75 - 30 - 12 / 2

= 39mm

Ast = 3 * (π * 12^2) / 4

= 339.3mm^2

γ_{min} = 1.05 - 0.007 * 40 (8.3.1(1))

= 0.77

α_2 = 1 - 0.003 * 40 (8.1.3(2))

= 0.85

ku $= \frac{1}{\alpha 2\gamma} + \frac{Ast}{bd} + \frac{fsy}{f}$

= 1 / (0.85 * 0.77) + 339.3 / (200 * 75) + 500 / 40

= 0.8308 > 0.36 **steel will not yield**

Compression controlled

c = ku d

= 0.8308 *39

= 32.4 mm (Neutral Axis)

a = γ ku d

= 0.77 * 32.4

= 24.95

M* $= 0.85 * f'c * b * a * \left(d - \frac{a}{2}\right)$

= 0.85 * 40 * 200 * 24.95 * (39 - 24.95 / 2)

= 4.500 kNm

ϕ = 0.6 ≤ (1.19 – 13ku / 12) ≤ 0.8 (Table 2.2.2)

= 0.29 ≤ 0.6 adopt 0.6

ϕM* = 4.500 * 0.6

= 2.700 kNm

Design Shear Strength

Vuc = $\beta 1\ \beta 2\ \beta 3\ bv\ do\ f'cv\ [\frac{Ast}{bv\ do}]^{1/3}$ (8.2.7.1)

β_1 = 1.1* (1.6 - d_o / 1000) $\geq$ 1.1

= 1.71

β_2 = 1

β_3 = 1

f'_{cv} = $f'c^{1/3} \leq$ 4Mpa (8.2.7.1)

= 3.42 Mpa

V* = 1.71 * 1 * 1 * 200 * 39 * 3.42 (339.3 / (200 * 39))$^{1/3}$

= 16.109 kN

ϕ = 0.7 (Table 2.2.2)

ϕV* = 0.7 * 16.109

= 11.276 kN

Sleeper 5 Calculations

Design Flexural Strength

d_o = 80 - 30 - 12 / 2

= 44 mm

Ast = 3 * (π * 12^2) / 4

= 339.3mm^2

γ_{min} = 1.05 - 0.007 * 40 (8.3.1(1))

= 0.77

α_2 = 1 - 0.003 * 40 (8.1.3(2))

= 0.85

ku $= \frac{1}{\alpha 2 \gamma} + \frac{Ast}{bd} + \frac{fsy}{f}$

$= 1 / (0.85 * 0.77) + 339.3 / (200 * 80) + 500 / 40$

$= 0.7364 > 0.36$ **steel will not yield**

Compression controlled

c $= \text{ku d}$

$= 0.8308 * 44$

$= 32.4$ mm (Neutral Axis)

a $= \gamma$ ku d

$= 0.77 * 32.4$

$= 24.95$

M* $= 0.85 * f'c * b * a * \left(d - \frac{a}{2}\right)$

= 0.85 * 40 * 200 * 24.95 * (44 - 24.95 / 2)

= 5.348 kNm

ϕ $= 0.6 \leq (1.19 – 13ku / 12) \leq 0.8$ (Table 2.2.2)

$= 0.392 \leq 0.6$ adopt 0.6

ϕM* = 5.348 * 0.6

= 3.209 kNm

<u>Design Shear Strength</u>

Vuc $= \beta 1\, \beta 2\, \beta 3\, bv\, do\, f'cv\, [\frac{Ast}{bv\, do}]^{1/3}$ (8.2.7.1)

β_1 $= 1.1 * (1.6 - d_o / 1000) \geq 1.1$

= 1.71

β_2 = 1

β_3 = 1

f'_{cv} $= f'c^{1/3} \leq 4$Mpa (8.2.7.1)

= 3.42 Mpa

V* $= 1.71 * 1 * 1 * 200 * 44 * 3.42 (339.3 / (200 * 44))^{1/3}$

= 17.402 kN

ϕ = 0.7 (Table 2.2.2)

ϕV* = 0.7 * 17.402

= 12.181 kN

Sleeper 6 Calculations

Design Flexural Strength

d_o = 75 - 25 - 10/2 = 45 mm

Ast = 2 * (π*10^2)/4 = 157.1 mm^2

γ_{min} = 1.05 - 0.007 * 50 = 0.70 (8.3.1(1))

α_2 = 1 - 0.003 * 50 = 0.85 (8.1.3(2))

Ku $= \frac{1}{\alpha 2\gamma} + \frac{Ast}{bd} + \frac{fsy}{f\prime c}$

= 1 / (0.85*0.70) + 157.1 / (150 * 75) + 500 / 50

= 0.3911 > 0.36 but < 0.4 **Assume steel yields**

Tension Controlled

k . do = 0.3911 *45 = 17.6 mm (Neutral Axis)

εst = 0.003/ k do * (do-k.do)

= 0.00467 (steel yields)

M* $= \text{ Ast fsy d } \left(1 - \frac{1}{2\alpha 2} + \frac{Ast}{bd} + \frac{fsy}{f'c}\right)$

= 157.1 * 500 * 45 (1 - 1 / (2* 0.85) + 157.1 / (150 * 45)) + 500/50

= 3.051 kNm

ϕ = 0.6 ≤ (1.19 – 13ku/12) ≤ 0.8 (Table 2.2.2)

= 0.766

ϕM* = 4.500 * 0.766

= 2.338 kNm

Design Shear Strength

Vuc = $\beta 1\ \beta 2\ \beta 3\ bv\ do\ f'cv\ [\frac{Ast}{bv\ do}]^{1/3}$ (8.2.7.1)

β_1 = 1.1* (1.6 - d_o / 1000) $\geq$ 1.1

= 1.71

β_2 = 1

β_3 = 1

f'_{cv} = $f'c^{1/3} \leq$ 4Mpa (8.2.7.1)

= 3.68 Mpa

V* = 1.71 * 1 * 1 * 150 * 45 * 3.68 $(157.1 / (150 * 45))^{1/3}$

= 12.14 kN

ϕ = 0.7 (Table 2.2.2)

ϕV* = 0.7 * 17.402

= 8.501 kN

Sleeper 7 Calculations

Design Flexural Strength

d_o = 100 - 30 - 16 / 2

= 62 mm

Ast = 2 * (π * 16^2) / 4

= 402.12 mm^2

γ_{min} = 1.05 - 0.007 * 40 (8.3.1(1))

= 0.77

α_2 = 1 - 0.003 * 40 (8.1.3(2))

= 0.85

ku = $\frac{1}{\alpha 2\gamma} + \frac{Ast}{bd} + \frac{fsy}{f}$

= 1 / (0.85 * 0.77) + 402.12/ (200 * 100) + 500 / 40

= 0.6194 > 0.36 **steel will not yield**

Compression controlled

c = ku d

= 0.6194 *62

= 38.4 mm (Neutral Axis)

a = γ ku d

= 0.77 * 38.4

= 29.57

M* $= 0.85 * f'c * b * a * \left(d - \frac{a}{2}\right)$

= 0.85 * 40 * 200 * 29.57 * (62 - 29.57 / 2)

= 9.493 kNm

ϕ = 0.6 ≤ (1.19 – 13ku / 12) ≤ 0.8 (Table 2.2.2)

= 0.519 ≤ 0.6 adopt 0.6

ϕM* = 9.493 * 0.6

= 5.696 kNm

<u>Design Shear Strength</u>

Vuc $= \beta 1\, \beta 2\, \beta 3\; bv\; do\; f'cv\, [\frac{Ast}{bv\; do}]^{1/3}$ (8.2.7.1)

β_1 = 1.1* (1.6 - d_o / 1000) ≥ 1.1

= 1.69

β_2 = 1

β_3 = 1

f'_{cv} = $f'c^{1/3}$ ≤ 4Mpa (8.2.7.1)

= 3.42 Mpa

V* = 1.69 * 1 * 1 * 200 * 62 * 3.42 $(402.1 / (200 * 62))^{1/3}$

= 22.879 kN

ϕ = 0.7 (Table 2.2.2)

ϕV* = 0.7 * 17.402

= 16.015 kN

Sleeper 8 Calculations

<u>Design Flexural Strength</u>

d_o = 100 - 30 - 16 / 2

= 62 mm

Ast = 3 * (π * 16²) / 4

= 603.12 mm²

γ_{min} = 1.05 - 0.007 * 40 (8.3.1(1))

= 0.77

α_2 = 1 - 0.003 * 40 (8.1.3(2))

= 0.85

ku $= \frac{1}{\alpha 2\gamma} + \frac{Ast}{bd} + \frac{fsy}{f}$

$= 1 / (0.85 * 0.77) + 603.12 / (200 * 100) + 500 / 40$

$= 0.9290 > 0.36$ **steel will not yield**

Compression controlled

c = ku d

= 0.9290 *62

= 57.60 mm (Neutral Axis)

a = γ ku d

= 0.77 * 57.6

= 44.35

M* $= 0.85 * f'c * b * a * \left(d - \frac{a}{2}\right)$

= 0.85 * 40 * 200 * 44.35 * (62 - 44.35 / 2)

= 12.011 kNm

ϕ = 0.6 ≤ (1.19 – 13ku / 12) ≤ 0.8 (Table 2.2.2)

= 0.184 ≤ 0.6 adopt 0.6

ϕM* = 12.011 * 0.6

= 7.206 kNm

Design Shear Strength

Vuc $= \beta 1\, \beta 2\, \beta 3\, bv\, do\, f'cv\, [\frac{Ast}{bv\, do}]^{1/3}$ (8.2.7.1)

β_1 = 1.1* (1.6 - d_o / 1000) $\geq$ 1.1

= 1.69

β_2 = 1

β_3 = 1

f'_{cv} = $f'c^{1/3} \leq$ 4Mpa (8.2.7.1)

= 3.42 Mpa

V* = 1.69 * 1 * 1 * 200 * 62 * 3.42 $(603.12\ /\ (200 * 62))^{1/3}$

= 26.190 kN

ϕ = 0.7 (Table 2.2.2)

ϕV* = 0.7 * 26.190

= 18.333 kN

5.3.5 Sleeper Adequacy Analysis

The results of the sleeper capacity are assessed against the design loading condition and the results tabled in the following section. Highlighted cells indicated failure of the sleeper in bending or shear. The maximum depth to which each sleeper can be used is therefore determined.

Sleepers 1, 2 and 3 (Design Condition)

Table 6: Sleeper 1, 2 & 3 capacity under design loading

Sleeper 1, 2 & 3						
Height	2m Spacing		1.5m Spacing		1.0m Spacing	
	*M	*V	*M	*V	*M	*V
0.2	1.639	3.279	0.922	2.459	0.410	1.639
0.4	2.33	4.651	1.308	3.488	0.581	2.326
0.6	3.01	6.024	1.694	4.518	0.753	3.012
0.8	3.70	7.396	2.080	5.547	0.925	3.698
1.0	4.38	8.769	2.466	6.577	1.096	4.384
1.2	5.07	10.141	2.852	7.606	1.268	5.071
1.4	5.76	11.514	3.238	8.635	1.439	5.757
1.6	6.44	12.886	3.624	9.665	1.611	6.443
1.8	7.13	14.259	4.010	10.694	1.782	7.129
2.0	7.82	15.631	4.396	11.723	1.954	7.816
2.2	8.50	17.004	4.782	12.753	2.125	8.502
2.4	9.19	18.376	5.168	13.782	2.297	9.188
2.6	9.87	19.749	5.554	14.812	2.469	9.874
2.8	10.56	21.12	5.94	15.84	2.64	10.56
3.0	11.25	22.49	6.33	16.87	2.81	11.25

Sleeper 4 (Design Condition)

Table 7: Sleeper 4 capacity under design loading

Sleeper 4						
Height	**2m Spacing**		**1.5m Spacing**		**1.0m Spacing**	
	***M**	***V**	***M**	***V**	***M**	***V**
0.2	1.64	3.28	0.92	2.46	0.41	1.64
0.4	2.33	4.65	1.31	3.49	0.58	2.33
0.6	3.01	6.02	1.69	4.52	0.75	3.01
0.8	3.70	7.40	2.08	5.55	0.92	3.70
1.0	4.38	8.77	2.47	6.58	1.10	4.38
1.2	5.07	10.14	2.85	7.61	1.27	5.07
1.4	5.76	11.51	3.24	8.64	1.44	5.76
1.6	6.44	12.89	3.62	9.66	1.61	6.44
1.8	7.13	14.26	4.01	10.69	1.78	7.13
2.0	7.82	15.63	4.40	11.72	1.95	7.82
2.2	8.50	17.00	4.78	12.75	2.13	8.50
2.4	9.19	18.38	5.17	13.78	2.30	9.19
2.6	9.87	19.75	5.55	14.81	2.47	9.87
2.8	10.56	21.12	5.94	15.84	2.64	10.56
3.0	11.25	22.49	6.33	16.87	2.81	11.25

Sleeper 5 (Design Condition)

Table 8: Sleeper 5 capacity under design loading

Sleeper 5						
Height	**2m Spacing**		**1.5m Spacing**		**1.0m Spacing**	
	***M**	***V**	***M**	***V**	***M**	***V**
0.2	1.64	3.28	0.92	2.46	0.41	1.64
0.4	2.33	4.65	1.31	3.49	0.58	2.33
0.6	3.01	6.02	1.69	4.52	0.75	3.01
0.8	3.70	7.40	2.08	5.55	0.92	3.70
1.0	4.38	8.77	2.47	6.58	1.10	4.38
1.2	5.07	10.14	2.85	7.61	1.27	5.07
1.4	5.76	11.51	3.24	8.64	1.44	5.76
1.6	6.44	12.89	3.62	9.66	1.61	6.44
1.8	7.13	14.26	4.01	10.69	1.78	7.13
2.0	7.82	15.63	4.40	11.72	1.95	7.82
2.2	8.50	17.00	4.78	12.75	2.13	8.50
2.4	9.19	18.38	5.17	13.78	2.30	9.19
2.6	9.87	19.75	5.55	14.81	2.47	9.87
2.8	10.56	21.12	5.94	15.84	2.64	10.56
3.0	11.25	22.49	6.33	16.87	2.81	11.25

Sleeper 6 (Design Condition)

Table 9: Sleeper 6 capacity under design loading

Sleeper 6						
Height	**2m Spacing**		**1.5m Spacing**		**1.0m Spacing**	
	***M**	***V**	***M**	***V**	***M**	***V**
0.2	1.64	3.28	0.92	2.46	0.41	1.64
0.4	2.33	4.65	1.31	3.49	0.58	2.33
0.6	3.01	6.02	1.69	4.52	0.75	3.01
0.8	3.70	7.40	2.08	5.55	0.92	3.70
1.0	4.38	8.77	2.47	6.58	1.10	4.38
1.2	5.07	10.14	2.85	7.61	1.27	5.07
1.4	5.76	11.51	3.24	8.64	1.44	5.76
1.6	6.44	12.89	3.62	9.66	1.61	6.44
1.8	7.13	14.26	4.01	10.69	1.78	7.13
2.0	7.82	15.63	4.40	11.72	1.95	7.82
2.2	8.50	17.00	4.78	12.75	2.13	8.50
2.4	9.19	18.38	5.17	13.78	2.30	9.19
2.6	9.87	19.75	5.55	14.81	2.47	9.87
2.8	10.56	21.12	5.94	15.84	2.64	10.56
3.0	11.25	22.49	6.33	16.87	2.81	11.25

Sleeper 7 (Design Condition)

Table 10: Sleeper 7 capacity under design loading

Sleeper 7						
Height	**2m Spacing**		**1.5m Spacing**		**1.0m Spacing**	
	***M**	***V**	***M**	***V**	***M**	***V**
0.2	1.64	3.28	0.92	2.46	0.41	1.64
0.4	2.33	4.65	1.31	3.49	0.58	2.33
0.6	3.01	6.02	1.69	4.52	0.75	3.01
0.8	3.70	7.40	2.08	5.55	0.92	3.70
1.0	4.38	8.77	2.47	6.58	1.10	4.38
1.2	5.07	10.14	2.85	7.61	1.27	5.07
1.4	5.76	11.51	3.24	8.64	1.44	5.76
1.6	6.44	12.89	3.62	9.66	1.61	6.44
1.8	7.13	14.26	4.01	10.69	1.78	7.13
2.0	7.82	15.63	4.40	11.72	1.95	7.82
2.2	8.50	17.00	4.78	12.75	2.13	8.50
2.4	9.19	18.38	5.17	13.78	2.30	9.19
2.6	9.87	19.75	5.55	14.81	2.47	9.87
2.8	10.56	21.12	5.94	15.84	2.64	10.56
3.0	11.25	22.49	6.33	16.87	2.81	11.25

Sleeper 8 (Design Condition)

Table 11: Sleeper 8 capacity under design loading

Sleeper 8						
Height	**2m Spacing**		**1.5m Spacing**		**1.0m Spacing**	
	***M**	***V**	***M**	***V**	***M**	***V**
0.2	1.64	3.28	0.92	2.46	0.41	1.64
0.4	2.33	4.65	1.31	3.49	0.58	2.33
0.6	3.01	6.02	1.69	4.52	0.75	3.01
0.8	3.70	7.40	2.08	5.55	0.92	3.70
1.0	4.38	8.77	2.47	6.58	1.10	4.38
1.2	5.07	10.14	2.85	7.61	1.27	5.07
1.4	5.76	11.51	3.24	8.64	1.44	5.76
1.6	6.44	12.89	3.62	9.66	1.61	6.44
1.8	7.13	14.26	4.01	10.69	1.78	7.13
2.0	7.82	15.63	4.40	11.72	1.95	7.82
2.2	8.50	17.00	4.78	12.75	2.13	8.50
2.4	9.19	18.38	5.17	13.78	2.30	9.19
2.6	9.87	19.75	5.55	14.81	2.47	9.87
2.8	10.56	21.12	5.94	15.84	2.64	10.56
3.0	11.25	22.49	6.33	16.87	2.81	11.25

5.4 Columns Analysis

5.4.1 Columns for Analysis

A variety of commonly used columns have been selected for analysis of their capacities when subject to the loading imposed by the design condition. The columns represent those utilised in the design alternatives along with multiple others for optimal design consideration. Parameters relating to the column properties have been sourced from OneSteel (OneSteel n.d) and are displayed below in table 12.

Table 12: Column properties for analysis

Columns (OneSteel)						
	d1 (mm)	tw (mm)	fsy (Mpa)	Ze 10^3mm^3	E (Mpa)	Ixx 10^6mm^4
100UC14.8	83	5.0	320	74	200	3.18
150UC23.4	139	6.1	320	176	200	12.60
200UB22.3	188	5.0	320	227	200	21.00
200UB25.4	188	5.8	320	259	200	23.60
250UB25.7	232	5.0	320	319	200	35.40
310UB32.0	282	5.5	320	467	200	63.20

5.4.2 Theoretical Loading Results

The maximum bending moment and shear force experienced by the column has been calculated in accordance with chapter 4 and displayed in table 13 below.

Table 13: Theoretical loading calculations for columns under design loading

Design Condition Factored Loading on Column						
Spacing:	**2.0**		**1.5**		**1.0**	
Height	**M***	**V***	**M***	**V***	**M***	**V***
0.2	0.09	1.04	0.07	0.78	0.05	0.52
0.4	0.45	2.62	0.34	1.97	0.23	1.31
0.6	1.18	4.76	0.89	3.57	0.59	2.38
0.8	2.39	7.44	1.79	5.58	1.20	3.72
1.0	4.19	10.68	3.15	8.01	2.10	5.34
1.2	6.70	14.46	5.02	10.84	3.35	7.23
1.4	10.01	18.79	7.51	14.09	5.01	9.39
1.6	14.25	23.67	10.69	17.75	7.12	11.83
1.8	19.52	29.10	14.64	21.82	9.76	14.55
2.0	25.93	35.08	19.44	26.31	12.96	17.54
2.2	33.58	41.60	25.19	31.20	16.79	20.80
2.4	42.60	48.68	31.95	36.51	21.30	24.34
2.6	53.09	56.30	39.82	42.23	26.55	28.15
2.8	65.16	64.48	48.87	48.36	32.58	32.24
3.0	78.92	73.20	59.19	54.90	39.46	36.60

5.4.3 Strand7 Loading Results

Strand7 finite element analysis has been used to verify the results of the loading condition on the column with respect to the maximum bending moment and shear force experienced by the columns. The following three scenarios were modelled for validation.

Table 14: Comparison of theoretical and modelling column results

Model No.	Height (m)	Spacing (m)	Theoretical M* (kN)	Model M* (kN)	Theoretical V* (kN)	Model V* (kN)
1	1	2	4.19	4.19	10.68	10.68
2	2	2	25.93	25.93	35.08	35.08
3	3	2	78.92	78.92	73.20	73.20

The results of analysis confirm that the theoretical calculations are correct and are suitable for use.

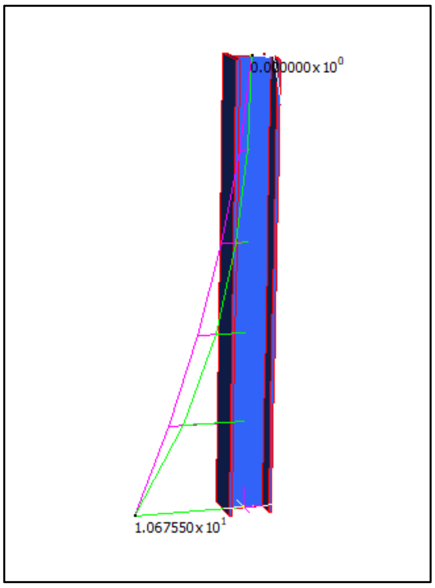

Figure 49: Column moment and shear model 1

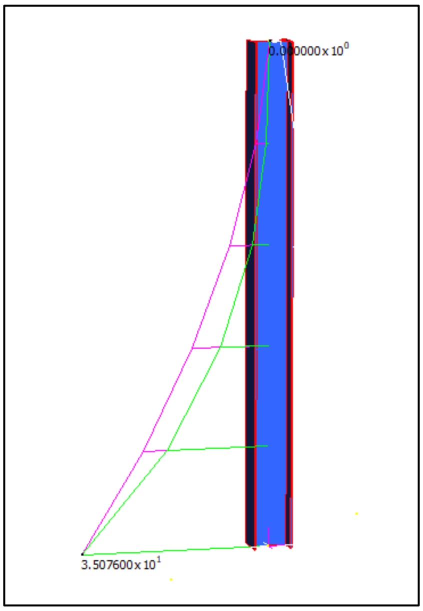

Figure 50: Column moment and shear model 2

Figure 51: Column moment and shear model 3

5.4.4 Column Capacities (Bending and Shear)

As identified in the previous methodology chapter, the columns are subjected to reduction factors in both the shear and flexural capacities.

100UC14.8

$\phi M = \phi \; fsy \; Zex$

ϕM = 0.9 * 320 * 74

= 21.312 kNm

$\phi V = 0.6 \; fy \; Aw$

ϕV = 0.6 * 320 * 83*5

= 79.680 kN

150UC23.4

$\phi M = \phi\ fsy\ Zex$

ϕM = 0.9 * 320 * 176

= 50.688 kNm

$\phi V = 0.6\ fsy\ Aw$

ϕV = 0.6 * 320 * 139 * 6.1

= 162.797 kN

200UC22.3

$\phi\mathrm{M} = \phi\ \mathrm{fsy\ Zex}$

ϕM = 0.9 * 320 * 227

= 65.367 kNm

$\phi V = 0.6\ fsy\ Aw$

ϕV = 0.6 * 320 * 188 * 5

= 180.480 kN

200UC25.4

$\phi M = \phi\ fsy\ Zex$

ϕM = 0.9 * 320 * 259

= 74.592 kNm

$\phi V = 0.6\ fsy\ Aw$

ϕV = 0.6 * 320 * 188 * 5.8

= 209.357 kN

250UC25.7

$\phi M = \phi \, fsy \, Zex$

ϕM = 0.9 * 320 * 319

= 91.872 kNm

$\phi V = 0.6 \, fy \, Aw$

ϕV = 0.6 * 320 * 232 * 5

= 222.720 kN

310UC32.0

$\phi M = \phi \, fsy \, Zex$

ϕM = 0.9 * 320 * 467

= 134.500 kNm

$\phi V = 0.6 \, fy \, Aw$

ϕV = 0.6 * 320 * 282 * 5.5

= 297.792 kN

5.4.4.1 Capacity Summary Table

Table 15: Column capacity for bending and shear forces

Column Capacity Summary		
Column	**ΦM (kNm)**	**ΦV (kN)**
100UC14.8	21.312	79.680
150UC23.4	50.688	162.797
200UB22.3	65.376	180.480
200UB25.4	74.592	209.357
250UB25.7	91.872	222.720
310UB32.0	134.496	297.792

5.4.5 Column Capacity (Serviceability)

A serviceability limit of 1:100 has been concluded to be appropriate for the deflection present at the free head of the column under load. The deflection for each column has been calculated and the results have been tabulated below according to column spacing. Highlighted cells identify the height at which the prescribed column will fail when loaded in accordance with the design condition.

5.4.5.1 2m Column Spacing

Table 16: Column capacity under design loading for 2m spacing

Column Deflection (mm) with 2m Spacing						
Height	100UC14.8	150UC23.4	200UB22.3	200UB25.4	250UB25.7	310UB32.0
0.2	0.00	0.00	0.00	0.00	0.00	0.00
0.4	0.03	0.01	0.00	0.00	0.00	0.00
0.6	0.15	0.04	0.02	0.02	0.01	0.01
0.8	0.54	0.14	0.08	0.07	0.05	0.03
1.0	1.47	0.37	0.22	0.20	0.13	0.07
1.2	3.34	0.84	0.51	0.45	0.30	0.17
1.4	6.75	1.70	1.02	0.91	0.61	0.34
1.6	12.45	3.14	1.89	1.68	1.12	0.63
1.8	21.46	5.42	3.25	2.89	1.93	1.08
2.0	35.01	8.84	5.30	4.72	3.14	1.76
2.2	54.63	13.79	8.27	7.36	4.91	2.75
2.4	82.14	20.73	12.44	11.07	7.38	4.13
2.6	119.71	30.21	18.13	16.13	10.75	6.02
2.8	169.86	42.87	25.72	22.89	15.26	8.55
3.0	235.49	59.43	35.66	31.73	21.15	11.85
3.2	319.94	80.75	48.45	43.11	28.74	16.10
3.4	426.97	107.76	64.66	57.53	38.35	21.48
3.6	560.81	141.54	84.92	75.57	50.38	28.22
3.8	726.21	183.28	109.97	97.85	65.24	36.54
4.0	928.43	234.32	140.59	125.10	83.40	46.72

5.4.5.2 1.5m Column Spacing

Table 17: Column capacity under design loading for 1.5m spacing

Post Deflection with 1.5m Spacing						
Height	**100UC14.8**	**150UC23.4**	**200UB22.3**	**200UB25.4**	**250UB25.7**	**310UB32.0**
0.2	0.00	0.00	0.00	0.00	0.00	0.00
0.4	0.02	0.01	0.00	0.00	0.00	0.00
0.6	0.11	0.03	0.02	0.02	0.01	0.01
0.8	0.41	0.10	0.06	0.05	0.04	0.02
1.0	1.10	0.28	0.17	0.15	0.10	0.06
1.2	2.51	0.63	0.38	0.34	0.23	0.13
1.4	5.06	1.28	0.77	0.68	0.45	0.25
1.6	9.34	2.36	1.41	1.26	0.84	0.47
1.8	16.09	4.06	2.44	2.17	1.45	0.81
2.0	26.26	6.63	3.98	3.54	2.36	1.32
2.2	40.97	10.34	6.20	5.52	3.68	2.06
2.4	61.60	15.55	9.33	8.30	5.53	3.10
2.6	89.78	22.66	13.60	12.10	8.07	4.52
2.8	127.39	32.15	19.29	17.17	11.44	6.41
3.0	176.62	44.58	26.75	23.80	15.87	8.89
3.2	239.96	60.56	36.34	32.33	21.56	12.07
3.4	320.23	80.82	48.49	43.15	28.77	16.11
3.6	420.61	106.15	63.69	56.68	37.78	21.16
3.8	544.66	137.46	82.48	73.39	48.93	27.41
4.0	696.32	175.74	105.44	93.83	62.55	35.04

5.4.5.3 1.0m Column Spacing

Table 18: Column capacity under design loading for 1m spacing

Post Deflection with 1.0m Spacing						
Height	100UC14.8	150UC23.4	200UB22.3	200UB25.4	250UB25.7	310UB32.0
0.2	0.00	0.00	0.00	0.00	0.00	0.00
0.4	0.01	0.00	0.00	0.00	0.00	0.00
0.6	0.08	0.02	0.01	0.01	0.01	0.00
0.8	0.27	0.07	0.04	0.04	0.02	0.01
1.0	0.73	0.19	0.11	0.10	0.07	0.04
1.2	1.67	0.42	0.25	0.23	0.15	0.08
1.4	3.37	0.85	0.51	0.45	0.30	0.17
1.6	6.23	1.57	0.94	0.84	0.56	0.31
1.8	10.73	2.71	1.62	1.45	0.96	0.54
2.0	17.50	4.42	2.65	2.36	1.57	0.88
2.2	27.31	6.89	4.14	3.68	2.45	1.37
2.4	41.07	10.37	6.22	5.53	3.69	2.07
2.6	59.85	15.11	9.06	8.07	5.38	3.01
2.8	84.93	21.43	12.86	11.44	7.63	4.27
3.0	117.75	29.72	17.83	15.87	10.58	5.92
3.2	159.97	40.37	24.22	21.56	14.37	8.05
3.4	213.48	53.88	32.33	28.77	19.18	10.74
3.6	280.41	70.77	42.46	37.78	25.19	14.11
3.8	363.11	91.64	54.98	48.93	32.62	18.27
4.0	464.21	117.16	70.30	62.55	41.70	23.36

5.4.6 Strand 7 Verification (serviceability)

Strand 7 has been used to verify the theoretical calculations for deflections. The analysis has been undertaken on the suitable member for the height of column as identified by the theoretical calculations. The following three scenarios were modelled for comparison.

Table 19: Comparison of theoretical and modelling deflection

Model No.	Height (m)	Spacing (m)	Column	Theoretical δ (mm)	Model δ (mm)
1	1	2	100UC14.8	1.47	1.59
2	2	2	150UC23.4	8.84	9.24
3	3	2	250UB25.7	21.15	22.00

The results show that the finite element modelling predicts a slightly higher deflection to be experienced by the column however the error is minor and considered acceptable for use when sizing a column.

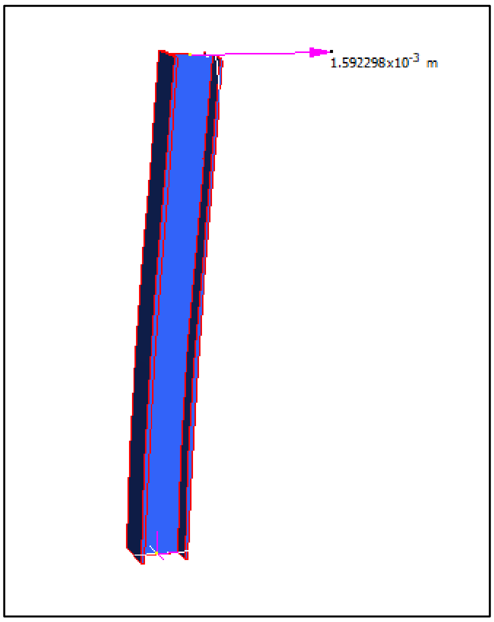

Figure 52: Column serviceability Model 1

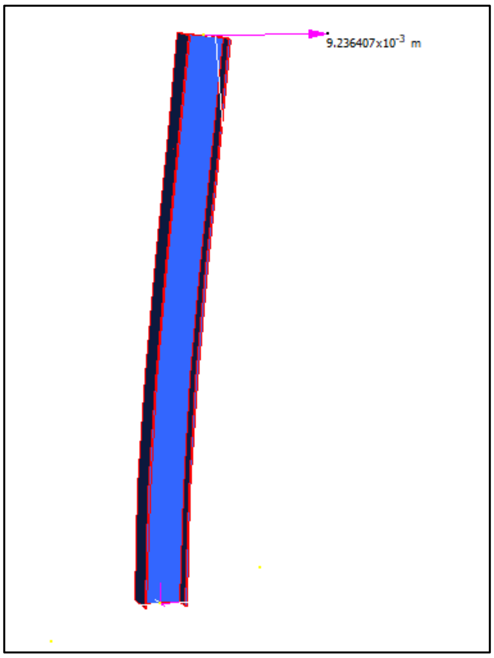

Figure 53: Column serviceability Model 2

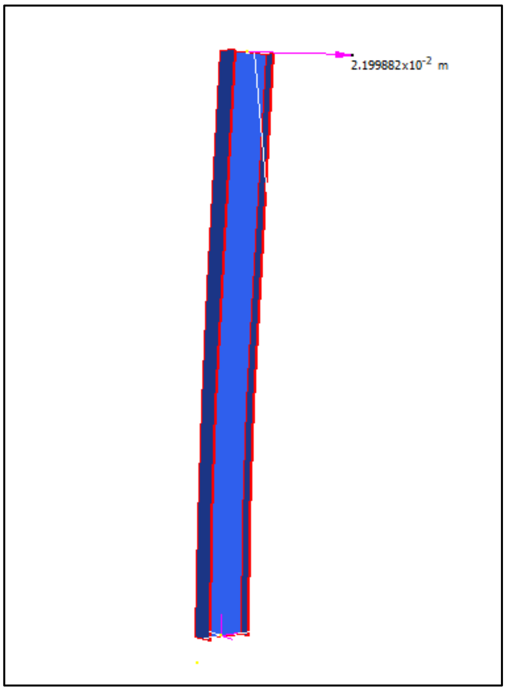

Figure 54: Column serviceability Model 3

5.5 Piers

The piers depth and maximum bending moment has been calculated in accordance with the methodology set out in chapter 4.

5.5.1 Required Depth

A factor of safety (FOS) of 1.5 has been applied to the passive earth pressure to determine the depth of pier required to prevent rotation. Pier diameters of 450mm and 600mm have been used and minimum embedment depth of 1m assumed. The calculated depths are displayed in table 20 below.

Table 20: Calculated required pier depths for design condition

Pier Design Depth - Cohesionless Soils						
	450 mm Pier			600mm Pier		
Height	**2m**	**1.5m**	**1.0m**	**2m**	**1.5m**	**1.0m**
0.2	1.0	1.0	1.0	1.0	1.0	1.0
0.4	1.0	1.0	1.0	1.0	1.0	1.0
0.6	1.4	1.2	1.2	1.4	1.2	1.2
0.8	1.6	1.4	1.4	1.6	1.4	1.4
1.0	2.0	1.8	1.6	2.0	1.6	1.6
1.2	2.4	2.0	2.0	2.2	2.0	2.0
1.4	2.8	2.6	2.2	2.4	2.2	2.2
1.6	3.0	2.8	2.4	2.6	2.4	2.4
1.8	3.4	3.0	2.6	3.0	2.6	2.6
2.0	3.6	3.2	2.8	3.2	2.8	2.8
2.2	4.0	3.4	3.2	3.4	3.2	3.2
2.4	4.2	3.6	3.4	3.8	3.2	3.4
2.6	4.6	4.0	3.6	4.0	3.6	3.6
2.8	4.8	4.2	3.8	4.2	3.8	3.8
3.0	5.2	4.6	4.0	4.6	4.0	4.0

5.5.2 Maximum Bending Moments

As stated in the design methodology, the columns are assumed to be monolithically cast into the pier to a depth of 100mm above the base. In this instance, the pier will have a flexural capacity equal to that of the column. The table below defines the maximum bending moment experienced by the pier and hence the column when subjected to the loading condition.

Table 21: Calculated maximum bending moment with the pier under design condition

Maximum Bending Moment (B) - Cohesionless Soils						
	450 mm Pier			600mm Pier		
Height	**2m**	**1.5m**	**1.0m**	**2m**	**1.5m**	**1.0m**
0.20	-	-	-	-	-	-
0.40	-	-	-	-	-	-
0.60	-	-	-	-	-	-
0.80	-	-	-	-	-	-
1.00	-	-	-	-	-	-
1.20	4.28	3.64	2.76	4.85	4.14	2.76
1.40	14.49	11.43	8.06	15.24	12.09	8.06
1.60	26.55	20.62	14.31	27.50	21.46	14.31
1.80	40.57	31.31	21.57	41.75	32.35	21.57
2.00	56.69	43.59	29.91	58.13	44.86	29.91
2.20	75.05	57.57	39.40	76.77	59.10	39.40
2.40	95.78	73.36	50.11	97.82	75.16	50.11
2.60	119.01	91.04	62.10	121.39	93.15	62.10
2.80	144.87	110.73	75.44	147.64	113.16	75.44
3.00	173.51	132.51	90.21	176.68	135.31	90.21

5.6 Conclusion

The results of the analysis have determined the maximum capacities for the sleepers, columns and piers with respect to strength, serviceability and failure.

The sleepers have been analysed in accordance with AS3600 and indicate that all sleepers, with the exemption of sleeper 6, have been over reinforced. The sleepers will subsequently experience a brittle failure and therefore are subject to an increased flexural reduction factor, (ϕ). This suggests that efficiencies in the geometric properties and area of reinforcement may be possible for this element.

The columns have been analysed for maximum shear, bending moment and serviceability in accordance with AS4100. A limit of 1:100 was used for the maximum deflection at the free head of the column. The serviceability limit is considered consistent with standard industry practice to maintain the aesthetic integrity of the wall.

The analysis of the pier capacity has yielded the required depths of piers to ensure soil failure and subsequent overturning does not occur. It is noted that the pier analysis is highly dependent on the insitu soil conditions and the results are indicative of the design condition only. As the column member acts against the bending moment within the pier, the maximum bending moment located at point B (see figure 39) needs to be considered when selecting the column member.

Chapter Six: Alternative Design Assessment

The following design alternatives have been tabled for uniformity and are a typical representation of designs used in South East Queensland. The designs have been attained via manufacturer websites along with typical engineered drawings that have been used on past projects. The designs will remain anonymous and are used solely as a comparative tool and an indication of the design discrepancy found in the industry.

6.1 Alternative Design 1

Design alternative 1 has been taken from a leading wall manufacturer design manual for a cohesionless granular soil and is considered suitable for comparison with the analysis results obtained in this study The highlighted cells show the capacity shortfalls in this design. It should be noted that again the sleeper capacity is far from sufficient in accordance with AS3600. The column sizing is consistent with the design methodologies proposed up to a height of 2.6m, thereafter the limits of serviceability proposed by this project would be exceeded. The pier depths are considered to be inadequate to prevent rotation in accordance with the methodology set out in chapter 4.

Table 22: Design alternative 1 analysis

Design Alternative 1					
Height	Spacing (m)	Sleeper (Type)	Column (Size)	Pier	
				dia (m)	depth (m)
0.2	2.0	1	100UC14.8	0.45	1.00
0.4	2.0	1	100UC14.8	0.45	1.00
0.6	2.0	1	100UC14.8	0.45	1.00
0.8	2.0	1	100UC14.8	0.45	1.00
1.0	2.0	1	100UC14.8	0.45	1.00
1.2	2.0	1	100UC14.8	0.45	1.20
1.4	2.0	1	100UC14.8	0.45	1.40
1.6	2.0	1	100UC14.8	0.45	1.60
1.8	2.0	1	150UC23.4	0.45	1.80
2.0	2.0	1	150UC23.4	0.45	2.00
2.2	1.5	1	150UC23.4	0.45	2.20
2.4	1.5	1	150UC23.4	0.45	2.40
2.6	1.5	1	150UC23.4	0.45	2.60
2.8	1.5	1	150UC23.4	0.45	2.80
3.0	1.0	1	150UC23.4	0.45	3.00

6.2 Alternative Design 2

Design alternative 2 has been based on a cohesionless granular soil and is therefore considered appropriate for comparison with the results yielded from the study analysis. Highlighted cells indicate insufficiencies in member capacity. It is noted that as with design alternative 1, the flexural capacity of the sleeper has shown to be insufficient, with sleeper 5 only providing a flexural capacity of 3.205 kNm. The column sizing is considered to be acceptable noting that due to unavailability of the internal angle of friction of the design strata it is plausible that the active pressure coefficient (Ka) could yield a lesser lateral earth pressure. The piers are considered to be insufficient for depth in a cohesionless soil when analysed using the proposed methodology set out in chapter 4.

Table 23: Design alternative 2 analysis

Design Alternative 2					
Height	Spacing (m)	Sleeper (Type)	Column (Size)	Pier	
				dia (m)	depth (m)
0.2	2.0	5	100UC14.8	0.45	0.60
0.4	2.0	5	100UC14.8	0.45	0.60
0.6	2.0	5	100UC14.8	0.45	0.60
0.8	2.0	5	100UC14.8	0.45	0.80
1.0	2.0	5	100UC14.8	0.45	0.95
1.2	2.0	5	100UC14.8	0.45	1.10
1.4	2.0	5	100UC14.8	0.45	1.30
1.6	2.0	5	100UC14.8	0.45	1.45
1.8	2.0	5	100UC14.8	0.45	1.60
2.0	2.0	5	150UC23.4	0.45	1.80
2.2	1.5	5	150UC23.4	0.45	1.80
2.4	1.5	5	150UC23.4	0.45	1.90
2.6	1.5	5	150UC23.4	0.45	2.00
2.8	1.5	5	150UC23.4	0.45	2.15
3.0	1.5	5	150UC23.4	0.45	3.00

6.3 Alternative Design 3

The engineering drawings associated with Alternative 3 do not define the type of soil to which the design is associated. As the design has been produced for a specific site address it is presumed the engineer has taken into account the localised soil conditions at the site. The author notes the doubling up of sleepers at a depth of 2.6m to achieve the desired flexural capacity. This is considered acceptable industry practice where both sleepers can fit inside the web of the column. In this case however the web depth of 139mm (150UC23.4) cannot accommodate the depth of 2 x Type 5 sleepers (160mm) and therefore it is assumed that the second sleeper would have to be cut short of the column flange. As such, while the flexural capacity of the sleeper may be enhanced, the shear strength is not and failure will be experienced at these depths (indicated in blue). When considered against the design condition the highlighted failures are evident.

Table 24: Design alternative 3 analysis

Design Alternative 3					
Height	Spacing (m)	Sleeper (Type)	Column (Size)	Pier	
				dia (m)	depth (m)
0.2	2.0	5	100UC14.8	0.45	1.00
0.4	2.0	5	100UC14.8	0.45	1.00
0.6	2.0	5	100UC14.8	0.45	1.00
0.8	2.0	5	100UC14.8	0.45	1.20
1.0	2.0	5	100UC14.8	0.45	1.40
1.2	2.0	5	100UC14.8	0.45	1.60
1.4	2.0	5	100UC14.8	0.45	2.00
1.6	2.0	5	100UC14.8	0.45	2.20
1.8	2.0	5	100UC14.8	0.45	2.40
2.0	2.0	5	100UC14.8	0.45	2.80
2.2	2.0	7	150UC23.4	0.45	3.00
2.4	2.0	7	150UC23.4	0.45	3.20
2.6	2.0	2 x 5	150UC23.4	0.45	3.40
2.8	2.0	2 x 5	150UC23.4	0.45	3.60
3.0	2.0	2 x 5	150UC23.4	0.45	3.80

6.4 Alternative Design 4

The wall has been designed to be constructed in stiff clay up to a maximum height of 2.4m. As the project analysis has been conducted on cohesionless soils (granular) the comparison of this design alternative with the design condition does not fairly assess the design. It should be noted that when using a cohesive soil in the estimation of earth pressures, the engineer needs to be particularly vigilant to ensure that the soil conditions are consistent in all areas across the site where the wall/s are situated. It can be seen in figure 13 that cohesive soil models predict that soils to a depth of ‘z’ do not act in an active manner against the wall, thus producing a reduced pressure at depth when compared to a cohesionless soil. This is a critical aspect of the bored pier retaining wall where local pier failure can result from discrepancies between bore hole surveys and insitu material.

Table 25: Design alternative 4 analysis

Design Alternative 4					
Height	Spacing (m)	Sleeper (Type)	Column (Size)	Pier	
				dia (m)	depth (m)
0.4	2.0	4	100UC14.8	0.45	0.8
0.6	2.0	4	100UC14.8	0.45	1
0.8	2.0	4	100UC14.8	0.45	1.2
1.0	2.0	4	100UC14.8	0.45	1.4
1.2	2.0	4	100UC14.8	0.45	1.6
1.4	1.5	4	100UC14.8	0.45	1.8
1.6	1.5	4	100UC14.8	0.45	2.0
1.8	1.5	4	100UC14.8	0.45	2.2
2.0	1.5	4	150UC23.4	0.45	2.4
2.2	1.5	4	150UC23.4	0.45	2.6
2.4	1.5	4	150UC23.4	0.45	2.8

6.5 Conclusion

The design alternatives have been assessed for adequacy against the methodologies for member capacity as described in chapter 4 of this study.

The alternatives show a lack of consistency in terms of element sizing and a trend of exceeding the capacity of members. It is noted that alternative design methodologies and serviceability limits may have been used by the engineer in determining the structural requirements of each member. However, it is reasonable upon review of the assessments that design of the flexural strength of sleepers is generally poor and that the depth of the pier is inconsistent and inadequate.

Chapter Seven: Wall Optimisation

7.1 Introduction

This chapter describes the optimisation of the bored pier retaining wall in cohesionless soils up to a maximum height of 3m. The wall is designed to provide adequate strength and stability properties while meeting the requirements set out in the following Australian Standards:

- AS3600 - Concrete Structures
- AS4100 - Steel Structures
- AS2159 - Piling- Design and Installation
- AS4678 - Earth Retaining Structures
- AS1170.0 - Structural Design Actions

The proposed design is consistent with that set out in chapter 4 in which the column is embedded monolithically with the pier construction to a depth of 100mm from the base of the pier.

7.2 Sleeper

The properties of the sleepers described in the previous section indicate that efficiencies can be made to provide adequate bending capacity and ductility. It is therefore proposed to provide a design using available sleepers as analysed in chapter 4 and an alternative sleeper design to achieve the performance required by these members.

7.2.1 Sleeper Design (Available Products)

Using the available sleepers that have been used for analysis in chapter 4, the following arrangement is proposed to satisfy the design condition. All reasonable effort has been made to reduce the variety of sleepers to assist with constructability. The available sleepers are detailed in table 3 in chapter 5.

Wall Height: 0 - 1.4m

Type 7 Sleeper

M* = 5.757 kNm

V* = 11.541 kN

ϕM = 5.646 kNm

ϕV = 16.051 kN (80mm depth)

Wall Height: 1.6 - 1.8m

Type 8 Sleeper

M* = 7.129 kNm

V* = 14.259 kN

ϕM = 7.206 kNm

ϕV = 18.333 kN (80mm depth)

Wall Height: 2.0m - 3.0m

Type 1 sleepers (back to back)

Sleepers are installed back to back to provide 150mm depth.

M* = 9.188 kNm

V* = 18.376 kN

ϕM = 10.014 kNm

ϕV = 19.661 kN (80mm depth)

7.2.2 Sleeper Design (Alternative Design)

The proposed alternative sleeper designs provide the required flexural capacity to sufficiently support the design load whilst maintaining a suitable depth at each end in order to fit the web of the detailed column.

In order to achieve the desired result a packer is to be inserted into the sleeper mould prior to pouring. Figure 55 details the desired effect of the packer, providing clearance of the web and flange at the rear of the wall. As identified earlier, the web depth is the key limiting factor for sleeper geometry. By implementing this arrangement, sufficient flexural strength can be achieved.

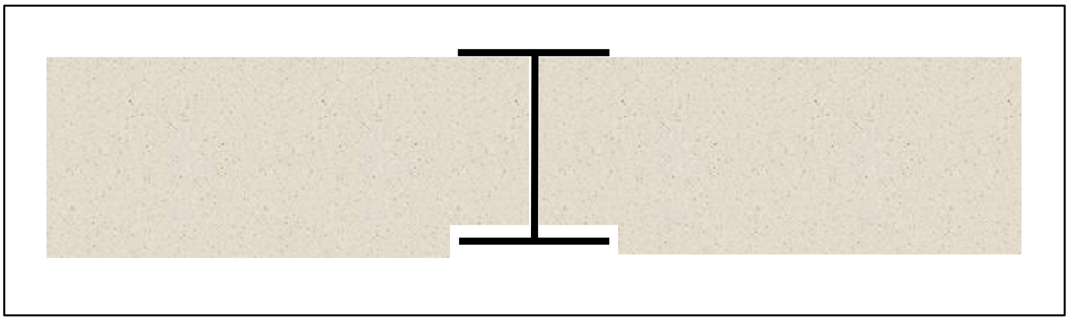

Figure 55: Proposed alternative sleeper design providing clearance of web depth

Wall Height: 0 - 1.0m

The sleeper has been designed to resist the nominal design condition from 0 - 1.0m where the maximum bending moment and shear force experienced are 4.384 kNm and 8.769 kN respectively (2m spacing). The shear capacity is governed by the minimum depth as defined by the packer width.

Properties:

d = 90mm

d = 200mm

Packer = 50mm x 200mm x 10mm

Reinforcement = 2 x N12

ku = 0.3661 < 0.4 (ductile)

ϕM = 4.548 kNm

ϕV = 11.396 kN (80mm depth)

Wall Height: 1.2m - 1.8m

The sleeper has been designed to resist the nominal design condition from 1.0m - 2.0m noting that the proposed spacing reduces from 2m to 1.5m at a height of 2.6m. The maximum bending moment and shear force is experienced by the sleeper occurs at a depth of 1.8m at 7.159 kNm and 14.259 kN respectively (2m spacing). The shear capacity is governed by the minimum depth as defined by the packer width.

Properties:

d = 115mm

d = 200mm

Packer = 50mm x 200mm x 35mm

Reinforcement = 3 x N12

Additional shear reinforcement: provide 1 x N12 @ 300mm run longitudinally at each end. (ie. 4 x N12 bars at maximum shear zone).

ku = 0.3857 < 0.4 (ductile)

ϕM = 9.369 kNm

ϕV = 14.359 kN (80mm depth)

Wall Height: 2.0m - 3.0m

The sleeper has been designed to resist the nominal design condition from 2.0m to 3.0m noting that the proposed spacing reduces from 2m to 1.5m at a height of 2.6m. The maximum bending moment and shear force is experienced by the sleeper occurs at a depth of 2.4m at 9.188 kNm and 18.376 kN respectively (2m spacing).

Properties:

d = 125mm

d = 200mm

Packer = NA

Reinforcement = 3 x N12

ku = 0.3447 < 0.4 (ductile)

ϕM = 11.064 kNm

ϕV = 19.557 kN (80mm depth)

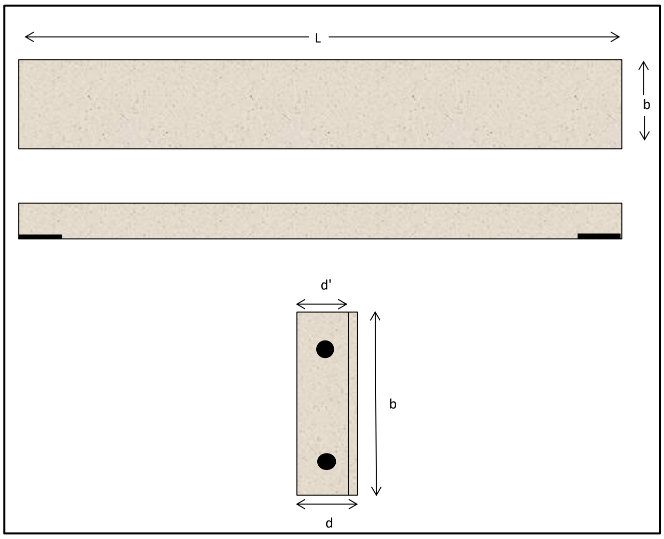

Figure 56: Geometric arrangement of design alternative

7.3 Columns

The columns were considered to be the most consistent aspect of the wall alternatives. The universal beams and columns have proven an adequate design selection for the walls and therefore have been maintained in the proposed wall optimisation. Table 26 sets out the proposed selection of columns with respect to wall height.

Table 26: Proposed column design

Proposed Column Design				
Height	**Spacing**	**M***	**V***	**Column**
0.2	2	0.095	1.037	**100UC14.8**
0.4	2	0.451	2.623	**100UC14.8**
0.6	2	1.180	4.758	**100UC14.8**
0.8	2	2.391	7.442	**100UC14.8**
1	2	4.194	10.675	**100UC14.8**
1.2	2	6.698	14.457	**100UC14.8**
1.4	2	14.490	11.430	**100UC14.8**
1.6	2	26.550	20.620	**150UC23.4**
1.8	2	40.570	31.310	**150UC23.4**
2	2	56.690	43.590	**200UB22.3**
2.2	2	75.050	57.570	**250UB25.4**
2.4	2	95.780	73.360	**250UB25.4**
2.6	1.5	39.819	42.227	**200UB22.3**
2.8	1.5	48.870	48.358	**200UB22.3**
3	1.5	59.189	54.900	**200UB22.3**

7.4 Piers

Much like the sleepers, the piers have shown to be inconsistently designed across the alternatives. The standard pier and foundation arrangement will be maintained for the purpose of constructability and economy. Table 27 below details the pier diameter and depth to satisfy the design condition. Piers are to be mass filled with N20 concrete.

Table 27: Proposed pier design

450mm Pier		
Height	**Spacing (m)**	**Required Depth**
0.2	2	1.0
0.4	2	1.2
0.6	2	1.4
0.8	2	1.6
1.0	2	2.0
1.2	2	2.4
1.4	2	2.8
1.6	2	3.0
1.8	2	3.4
2.0	2	3.6
2.2	2	4.0
2.4	2	4.2
2.6	1.5	4.0
2.8	1.5	4.2
3.0	1.5	4.6

Chapter Eight: Conclusions and Recommendations

8.1 Conclusions

Earth retention structures are a versatile and necessary construction element used across multiple construction industries. Notably, the bored pier retaining wall features prominently in civil land development projects throughout South East Queensland and provided the key focus for this project. Through the analysis conducted in this report, awareness can be raised regarding the lack of consistency in design. This dissertation is designed to provide a reference to improve the level of design and construction relating to these walls.

The research presents the key theories, parameters and equations relating to wall design. It discussed the common walls used in the civil construction industry in South East Queensland and analysed the benefits, costings and construction methodologies used for a number of these.

Justification regarding the requirement for the bored pier retaining wall to undergo further assessment was presented and included a gap in theoretical knowledge, a high failure rate in the industry and clear lack of industry confidence in the product.

The presented methodologies provided the basis for a comparison of acquired design alternatives which in turn identified large discrepancies in the sizing of members and perceived lack of industry understanding relating to earth pressure and member capacity.

An alternative design was presented in accordance with Australian Standards. The proposal utilised off the shelf products as well as an alternative sleeper design to achieve ductile, tension controlled members.

Despite a proven lack of industry confidence, bored pier retaining walls continue to feature prominently in civil land development projects. The study concludes that by adopting the recommendations outlined below, design engineers will provide a more stable and reliable earth retention structure.

8.2 Recommendations

The dissertation has identified a number of key recommendations to be proposed to industry in order to unify the understanding of requirements relating to the bored pier retaining wall.

1. A clear industry expectation of the requirement of ductile failure for the reinforced concrete sleepers should be determined.

2. A standardised design methodology should be adopted to ensure safe and unified design practices for the required pier diameter and depth.

3. Rankine's theory of lateral earth pressures should be adopted in order to avoid the underestimation of loadings for bored pier walls.

4. Adopting cohesive soil parameters should be approached with caution, particularly on large sites. An engineer's reliance on contractor feedback regarding localised site conditions is not considered adequate. The designer should therefore allow for additional site visits where conservative earth pressures are used.

5. Interpretation of geotechnical surveys to determine the soil parameters for which a retaining wall is to be designed should be undertaken by the project consulting engineer only, rather than the design and construct contractor.

6. It is suggested that the head contractor have a greater responsibility with regard to adequate construction practices by design and construction subcontractors.

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Appendix A - Project Specification

Dissertation Specification

For: Simon Stewart

Title: Future Trend of Retaining Walls in South East Queensland

Major: Civil Engineering

Supervisors: Weena Lokuge

External Supervisor: Ching Man Tan (Principal Civil Engineer)

Enrolment: ENG4111 EXT. S1, 2017

ENG4112 EXT, S2 2017

Project Objectives:

1. Provide a review of retaining wall design and recent changes and trends seen in South East Queensland.
2. Provide a detailed analysis of retaining walls commonly used in South East Queensland urban development including their common application and detailed cost feasibility studies.
3. Analyse the properties, parameters and equations used for the identified common retaining wall design.
4. Identify an appropriate commonly used retaining wall system and undertake detailed structural analysis of the design variations commonly used in South East Queensland.
5. Provide recommendations based on research and design analysis on the best practice construction methodology and design for the above identified wall.

Programme:

1. Undertake review on the history of retaining wall design in Australia and conduct a survey of industry professionals to identify common practice and trends.
2. Literature review and analysis of the properties, parameters and equations used for the identified common retaining wall design.
3. Conduct a cost feasibility analysis of the commonly used walls and their suitability to performance, aesthetic and in-situ conditions.
4. Undertake a structural analysis of an appropriate retaining wall of interest commonly used in in South East Queensland urban development including comparison of design variations.
5. Draw conclusions and recommendations based on the above.
6. Prepare and present presentation, finalise report and submit prior to 12th October, 2017.