UNIVERSITY OF SOUTHERN QUEENSLAND

School of Civil Engineering and Surveying

Analysing Geotechnical Aspects of Concrete Pipe Culverts

A dissertation submitted by

Simon James Porter

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Abstract

Concrete pipe culverts are important structures used throughout the world to convey water and provide access beneath roadways. Culvert design requires an understanding of the structural, hydraulic, construction and geotechnical aspects that influence the functioning of the structure. Geotechnical aspects of culvert structures significantly affect the design and include factors like bedding and backfill type, installation conditions and loading analysis. A disparity exists between current design methodology and in field observations. Such design methods are affected by certain assumptions which may not be met in field during construction due to site restrictions, cost, material availability or other factors.

The impact of analysis technique for assessing the distribution of live and dead loads of soil on the pipe, as well as the soil structure interaction, can significantly affect the design. Some of these techniques do not adequately provide for the assessment of alternative construction methods or backfill materials. Therefore, the impact of these changes may not match the impact assessed in the design. There is the potential for modern Finite Element Method (FEM) programs to more accurately evaluate culvert loadings and describe alternative construction conditions. This research has been prepared in order to compare the various analysis methods within FEM packages, as well as the current design standard, while also assessing the contribution that changed construction methods have upon culvert loading. This involved outlining design scenarios for the trench condition with varying trench widths, backfill heights (ranging from 0.3 m to 1.2 m) and bedding and backfill materials (including conforming granular backfill, non-conforming granular backfill, aggregate, stabilised sand and controlled low strength materials (CLSM)). These scenarios have been analysed utilising the Culvert Analysis and Design (CANDE) FEM program, with a linear elastic, Mohr Coulomb and Duncan soil model, as well as by the Australian/New Zealand Standard *Design for installation of buried concrete pipes* (AS/NZS 3725). These techniques were then used to assess alternative construction methods in industry for two case studies. One was assessing aggregate backfill while the other was assessing the use of stabilised sand for low cover applications.

The findings identified that the Australian Standard design method resulted in significantly reduced factors of safety for alternative bedding and backfill materials, whereas in some cases the CANDE models observed the opposite result. However, for conforming granular materials meeting HS3 support conditions the AS/NZS 3725 method resulted in higher safety factors than the FEM program. The AS/NZS 3725 design method also significantly underestimated the capacity of pipes for low covers (below 400 mm) in comparison to a FEM analysis method.

Drawbacks of the FEM models were identified with the Mohr Coulomb model, which resulted in non-convergence for non-plastic materials with low shear strength, due to the high surface loading. The Duncan model, while being identified as the most realistic model (of the models compared), has limitations for assessing alternative materials as the model requires specific parameters based upon the material properties.

A comparison of the scenarios identified that the stabilised sand and CLSM materials could result in potentially greater safety factors against failure, as well as reduced displacements, in comparison to the other materials. Aggregate backfill also had the potential to reduce the loading upon the pipe and displacements of the backfill, however, this is significantly affected by construction processes. The non-conforming materials performed similarly to the conforming granular materials, however, those with higher clay content generally exhibited higher displacements. There are also issues associated with the response of such material to moisture. For the cover and trench widths assessed, it was determined that narrower trenches were favourable for the material types, however, an ability to adequately compact the material needs to be maintained. Finally, for the highway loadings utilised it was identified that the deeper cover depths were preferable to the shallow cover depths due to the increased load distribution outweighing the disadvantage of increased dead loading.

Analysis of the case studies revealed that the alternative materials performed better than the conforming material when using FEM modelling. However, this was inconsistent with the standard method which identified that particular alternative materials may be inadequate. The standards results were also inconsistent with the field performance of these culverts.

The improved understanding of the performance of various construction methods can allow for better decision making in the field. From the results of the research, superior (or in some cases similar) performance can be achieved by alternative or non-conforming backfill in comparison to conforming materials. Conservative design processes are apparent in some aspects of the current standard design method which can be improved through the use of more accurate FEM models. Future research analysing the field performance of these alternative construction techniques is required in order to assess the FEM programs ability to analyse the material. An expansion on the range of material properties available to the Duncan model could also improve the models ability to analyse alternative backfill materials.

Declaration

University of Southern Queensland

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ENG4111/ENG4112 Research Project

Certification 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.

S. Porter

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Nomenclature

This list comprises the primary symbols utilised within the document and those which are not otherwise defined within the text. Symbols not identified within this list have been identified within the relevant section.

ϕ - Friction Angle

c - Cohesion

γ - Unit Weight

E – Young's Modulus

ρ – Density

G – Shear Modulus

σ – Stress

ε – Strain

τ – Shear Stress

ν – Poisson's Ratio

λ – Lamê Modulus

K – Dimensionless Magnitude of Initial Young's Modulus

n – Power Law Coefficient for Initial Modulus

ϕ_0 – Reference Soil Friction Angle

$\Delta\ \phi$ – Reducion of Friction Angle

R_f – Reduction Factor

K_b – Dimensionless Magnitude of Tangent Bulk Modulus

m – Power Law Coefficient

B_i – Initial Bulk Modulus

P_a – Atmospheric Pressure

ε_u – Ultimate Volumetric Strain at Large Hydrostatic Stress

f'c – Specified Compressive Strength

D_{max} – Maximum Diameter of Particle

D_{50} – Diameter at which 50% of the material passes

C_u – Uniformity Coefficient

C_c – Coefficient of Gradation

1. Introduction

1.1 Project Background

Culverts are integral roadway structures that provide a passage for water to move through a road corridor. They also have alternative functions as wildlife and fish passages and potentially pedestrian, cyclist and vehicular access ways (TMR, 2013). The structures are designed to withstand vehicular and soil loading as well as provide adequate flood resistance and limit the potential ingress of water into the pavement and bedding. Such ingress and moisture fluctuations generally lead to pavement and culvert degradation which may result in costly remediation (Wagener & CNAConsultingEngineers, 2014). Inadequate consideration of structural requirements can also cause culvert collapse, infrastructure damage, erosion and sedimentation of water bodies as well as impact upon the health and safety of the road and culvert users.

The performance of culvert structures is important to the safety of road users and the performance of roadways (Beaver, McGrath, & Leonard, 2004; Tran, 2014). Therefore development to improve the reliability of design and implementation is important. Culvert replacements can be costly and these structures should be designed effectively to meet an economical lifespan. As such, continued development and review of design practices is necessary in order to improve performance. This project involved assessing the ways in which current design standards model culvert-soil interactions. It also compared existing culvert design methods to assess current trends and assumptions while analysing potential onsite variations to these assumptions.

This report has been provided to improve the current understanding and develop the field of culvert design. A literature review is presented in order to identify the current status of research in the area of the geotechnical aspects of culvert structures. The literature review was also a key part in refining and developing the aims, objectives and employed methodologies utilised within the project. The methodology was planned in order to deliver effective and useful project outcomes while minimising the risk and ensuring the project resourcing and timing restrictions were met. The results have been outlined and their implications discussed in order to identify relevant conclusions that meet the aims of the project.

1.2 Project Aims and Objectives

This research involved analysing the geotechnical aspects of the construction of concrete pipe culverts. The project was specifically developed due to the practical benefits of an improved understanding of culvert-soil interactions in industry. It is recognised that there is significant potential for failure when constructing subsurface structures that alter the existing ground conditions and which are subject to various loading conditions (Tran, 2014). In particular, pipe construction methods can have a major impact on the potential load distribution to the structure and the movement of soil. Therefore, an understanding of design practices and assumptions is necessary to ensure that the design meets the in field construction methods (Tran, 2014; Yoo, Parker, & Kang, 2005). Trench design is an important area which influences the cost of the structure, safety of the construction process, potential environmental impacts as well as the loading on the structure (Chen & Sun, 2014; Yoo et al., 2005).

The topic for the project arose while working as an undergraduate engineer in a project management role with the Roads and Maritime Services (RMS). During this role experience was gained analysing the condition of existing network assets and targeting maintenance programs to rectify existing defects. This work led to identifying that there were many failures surrounding culverts which were often high priority due to the potential flow-on effects on pavement condition as well as road user safety. Often such defects, if not rectified early, would require costly remediation techniques. When exposed to several such remediation projects it was noticed that similar repairs were often undertaken by different project teams utilising different methods to obtain similar results. Hence, the importance of understanding the geotechnical impacts of culverts and the potential success of remediation methods was emphasised early.

Through communication between various project managers, asset managers and geotechnical scientists it was determined that an improved understanding of the techniques currently utilised for culvert design and remediation was necessary. A journal article by Tran (2014) identified variations between culvert design techniques with the Australian Standard utilising the indirect analysis method which is well established. However, it was noted that developments in direct analysis methods enabled more accurate modelling of the soil structure interactions and pipe stress. Yoo et al. (2005) also assessed certain deficiencies in the American Association of State Highway and Transportation Officials (AASHTO) design guides through a theoretical assessment of soil structure interaction, trench design and construction techniques.

Therefore the initial aim was to compare and assess the effects of construction and design methods for concrete pipe culverts. In order to achieve this, comparisons were proposed to analyse specific culvert backfill and bedding types which was aimed to contribute to the understanding of effective culvert design and construction methods. The focus of the research was specifically on the geotechnical analysis and design and how this affects loading conditions, stresses, strains and displacements for culvert structures and the supporting material.

The broad objectives of the project have been developed with the previously outlined issues of culvert analysis, design and construction in mind. These objectives are as follows

1) Research the various materials and methods utilised for placing the bedding and backfilling pipe culverts.
2) Research the various analysis techniques for assessing the load distribution upon pipe culverts in various conditions.
3) Determine several scenarios to compare individual culvert construction and load assessment measures.
4) Identify best case scenarios and limitations of the various methods as well as assessing the current safety factors in design.
5) Collect and compile current culvert inventory data on reinforced concrete pipe culverts (RCPC) within the Northern New South Wales (NSW) RMS region to assess the causes of existing failures.
6) Analyse the contribution that construction methods and geotechnical conditions make to the failures identified within RMS Northern region.

Meeting these aims and objectives improved upon the existing understanding of culvert design. This information was proposed to be utilised to assist designers and project engineers who utilise current design standards and methods, when making decisions on construction methods and when deviating from planned methods.

The expected outcomes of the project were to:

- determine the variation between analytical, empirical and numerical design methods;
- assess the implications of these variations;
- provide an assessment of how existing culverts have been impacted by potential deviations from standard methods;
- determine the most suitable construction methods and design techniques.

The assessment of current design techniques has the potential to improve design practices and implementation. This would provide broad benefits of cost saving by improving the potential lifespan of culvert structures. Improved safety and environmental benefits of correctly functioning culverts may also result from continual improvement and analysis of design practices. Useful information regarding current design software was also collected during this comparison.

2. Literature Review

2.1 Introduction

Culverts are drainage structures which provide passage for water through a road corridor, limit flooding potential and deliver various other functions such as fish and fauna passage, fauna habitat, passageways and support structures (TMR, 2013). The primary function of moving water through a road corridor is to ensure that water infiltration into pavement materials is limited. This is required due to the negative effects that moisture can have on the pavement structure which include; reducing the stability of unbound pavement, decreasing bearing capacity and increased shearing potential, rutting, delamination, loss of fine particles and moisture damage to bituminous materials (Australian Road Research Board, 2003; Choi, 2007). Given the importance of culvert structures to ensure the correct functioning of road networks and storm water systems, there has been considerable development in the fields of culvert design, maintenance requirements and repair methods, as well as alternative construction methods and culvert types. Along with concrete box culverts, RCPC have historically been the predominant culvert type in Australia (TMR, 2013) and as such will be the predominant focus of this research. This preference is due to the ability of rigid culverts to resist forces applied by earth pressures, soil weight, traffic and construction loading through the materials structural strength. The rigid structure has greater stiffness than the surrounding soil and therefore carries the majority of the load (ConnDOT, 2000; NYSDOT, 2013).

The mechanism of load transfer through the soil/backfill material is an important factor in reducing stresses applied to culvert structures. The reduction is due to the increase in area over which a load will act in a granular material with depth. Calculating this change in stress with depth is generally achieved by making assumptions that the soil is elastic, isotropic and homogenous or by an empirical estimate of load reduction (Das, 2010), however these assumptions often do not hold true. Variation between analysis methods is observed and was analysed within this study. Improved computing power has also allowed for the development of software packages that can discretise the soil structure into segments with varying properties that can simulate inelastic and inhomogeneous load response. The analysis of the soil structure interaction is also an important part of this process in order to consider the interaction of the backfill material with the pipe (Yoo et al., 2005).

Extensive research has been carried out on the structural, hydraulic, construction and geotechnical aspects of culvert structures in order to develop effective design requirements. In

Australia this research has led to the formation of Australian Standards and state government transport bodies' specifications and manuals. In particular there are Australian Standards that govern the structural and geotechnical requirements of the construction of culvert structures. AS/NZS 3725-2007 *Design for Installation of Buried Concrete Pipes* (Committee:WS-006, 2007a) with reference to AS 5100.2 *Bridge Design: Design Loads-2004* (Committee:BD-090, 2004) outline required standards for concrete pipe culverts in Australia. These requirements include specific bedding and backfill types, defining specific pipe support conditions and determining the design loadings to be considered (Committee:WS-006, 2007a). In NSW the state transport body is the RMS and their main quality assurance document is the specification for Stormwater and Drainage Structures R11 (RMS, 2013). Other documents are also offered by interstate road authorities, which will be assessed and compared in conjunction with the RMS specification. Such documents outline specific construction methodologies and material requirements. The AASHTO also provide standards for the determination of culvert loading which will be considered within this research. AASHTO's methods only consider soil density rather than the potential support provided from the soil. This does not accurately reflect the conditions that occur in reality but provides a conservative approach to design (Jayawickrama, Senanayake, Lawson, & Wood, 2012; Wood, Lawson, Newhouse, & Jayawickrama, 2014; Yoo et al., 2005). This chapter aims at identifying the various assumptions and theories as well as researching the construction practices that effect the design requirements.

2.2 Bedding and Backfill Methods

Variation in culvert backfill types will alter the soil-structure interaction and loading conditions upon the pipe (Rajah, McCabe, & Plattsmier, 2012; Tysl & Noll, 2011; Yoo et al., 2005). The effect of backfill variation on current and alternative design practices is therefore of importance. The use of various backfill types is justified by availability, price and constructability. These include utilising a granular backfill, controlled low strength material (CLSM) and open graded stone.

2.2.1 Gravel Backfill

The RMS specification R11 states requirements for bedding and backfill material based upon particle size distribution (PSD), plasticity index (PI) and maximum particle size (RMS, 2013). The PSD in R11 references AS/NZS 3725 for the bed and haunch zones, however the side and overlay zones definition varies from the Australian Standards grading and maximum size limits to outline only a maximum size and plasticity (definitions for material zones are shown in Figure 2.1) (Committee:WS-006, 2007a; RMS, 2013).

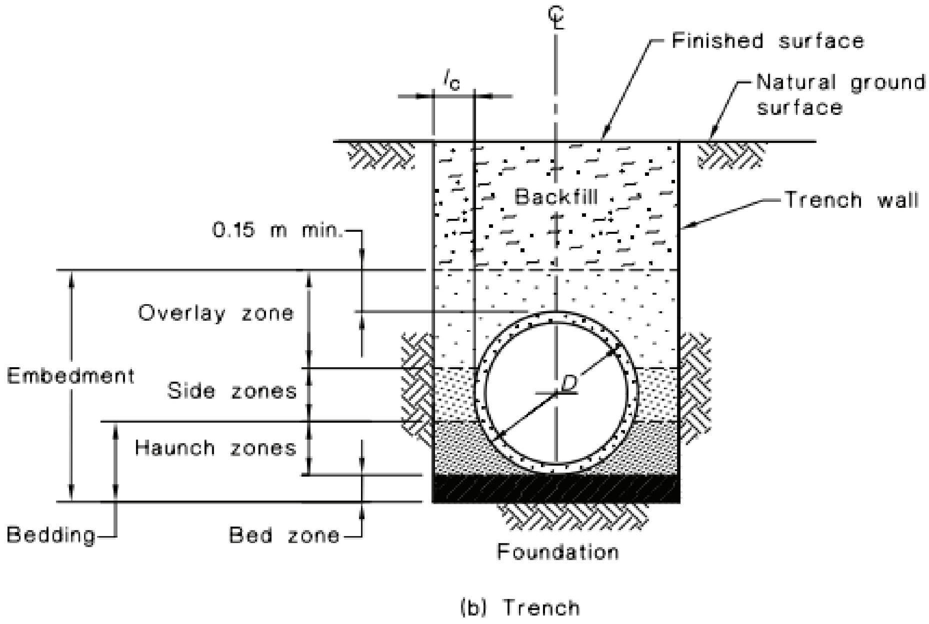


Figure 2.1: Fill and Pipe Support Terms (Committee:WS-006, 2007a)

R11 defines two granular material types for culvert backfill which are Type BH and SO. BH material is utilised for the bed and haunch zones while SO is for side and overlay zones. Type BH is required to conform to a PSD as set out by AS/NZS 3725 (Committee:WS-006, 2007a) (Table 2.1) and a PI not more than 6. However, type SO is required only to have a maximum particle size no greater than 50 mm and a PI between 2 and 12 (RMS, 2013).

Table 2.1: Grading Limits for Select Fill (Committee:WS-006, 2007a)

Sieve Size (mm)	Bed and Haunch (BH) Zones Percent by weight passing (%)	Side Zones Percent by weight passing (%)
75		100
19	100	
9.5		100-50
2.36	100-50	100-30
0.6	90-20	50-15
0.3	60-10	
0.15	25-0	
0.075		25-0

NOTE: Acceptable material within the above grading limits would result in material that is well graded and free draining. Granular material that may exhibit these qualities but would break down when wetted such as shale or gravely conglomerates are not suitable materials and shall not be used.

These material properties can be utilised in conjunction with the Unified Soil Classification System (USCS) in order to determine the applicable soil types for use as a conforming backfill material. A range of material properties for specific soil classifications can be found within the literature to make reliable comparisons of material properties and the effect of these properties on culvert structures. In particular it was found that the grading limits and other requirements for type BH material can be classified as SW (Well graded sand) and SM (Silty sand) while type SO can be classified as SW, SM and SC (Clayey sand) (Das, 2010). Standard material properties for USCS classified gravels are shown in Table 2.2 with properties relating to the Duncan Selig soil model and the Duncan soil model given in Table 2.3 and Table 2.4 respectively. For comparison, general ranges for elastic material properties were given in Table 2.5.

Table 2.2: USCS Material Properties (Rajah et al., 2012)

Soil Description		**Manufactured Stone [1]**	**Coarse Grained Soil**				**Fine Grained Soil**		
			Little or No Fines		With Fines		Low Plasticity	High Plasticity	Organic Silt, Clay, Peat
Proposed Soil Groups		I	II	III	IV	V	VI	VII	VIII
Soil Classification	USCS [2]	GW, GP	GW, GP	SW, SP	GM, GC	SM, SC	ML, CL	MH, CH	OL, OH, PT
<=85% Proctor[4]	Friction Angle (ϕ) deg	36.00	34.00	31.00	30.00	30.00	30.00	28.00	See Note 3
	Cohesion (c) kPa	0.00	0.00	0.00	0.00	9.58	4.79	9.58	See Note 3
	Unit Weight (γ) kN/m³	20.43	18.07	18.07	17.29	17.29	17.29	17.29	See Note 3
	E, MPa	6.89	4.83	4.83	4.14	4.14	3.45	3.45	See Note 3
85-90% Proctor[4]	Friction Angle (ϕ) deg	38.00	35.00	32.00	31.00	31.00	30.00	28.00	See Note 3
	Cohesion (c) kPa	0.00	0.00	0.00	0.00	14.36	9.58	14.36	See Note 3
	Unit Weight (γ) kN/m³	20.43	18.86	18.86	17.91	17.91	17.60	17.60	See Note 3
	E, MPa	10.34	6.89	6.89	6.89	6.89	4.83	4.83	See Note 3
90-95% Proctor[4]	Friction Angle (ϕ) deg	38.00	35.00	34.00	32.00	32.00	30.00	28.00	See Note 3
	Cohesion (c) kPa	0.00	0.00	0.00	0.00	16.76	11.97	16.76	See Note 3
	Unit Weight (γ) kN/m³	20.43	19.17	19.17	18.23	18.23	17.76	17.76	See Note 3
	E, MPa	17.24	8.96	8.96	7.58	7.58	5.86	5.86	See Note 3
>95% Proctor[4]	Friction Angle (ϕ) deg	39.00	36.00	36.00	33.00	33.00	30.00	28.00	See Note 3
	Cohesion (c) kPa	0.00	0.00	0.00	0.00	19.15	14.36	19.15	See Note 3
	Unit Weight (γ) kN/m³	21.21	19.64	19.64	19.64	18.86	18.07	18.07	See Note 3
	E, MPa	20.68	11.72	11.03	11.03	8.27	6.89	6.89	See Note 3

Notes:

1. Manufactured stone: Angular crushed gravel/stone, possibly sand
2. Unified Soil Classification System
3. Not suitable as pipe zone backfill
4. Standard compaction effort, per ASTM D698. Percent std proctor values may be obtained by adding 5% to % proctor density values obtained per ASTM D1557

Table 2.3: Duncan Selig Soil Properties (Katona, 2015)

Soil Type and Compaction	Young's Tangent Modulus Parameters						Bulk Parameters		Density reference (kg/m3)
	K (--)	n (--)	c (Pa)	Φ_0 (deg)	$\Delta\phi$ (deg)	R_f (--)	B_i/P_a (--)	ε_u (--)	
SW100	1300	0.90	0	54	15	0.65	108.8	0.01	23.25
SW95	950	0.60	0	48	8.0	0.70	74.8	0.02	22.78
SW90	640	0.43	0	42	4.0	0.75	40.8	0.05	21.99
SW85	450	0.35	0	38	2.0	0.80	12.7	0.08	20.42
SW80	320	0.35	0	36	1.0	0.90	6.1	0.11	18.85
ML95	440	0.40	28	34	0.0	0.95	48.3	0.06	21.21
ML90	200	0.26	24	32	0.0	0.89	18.4	0.10	20.42
ML85	110	0.25	21	30	0.0	0.85	9.5	0.14	19.17
ML80	75	0.25	17	28	0.0	0.80	5.1	0.19	18.07
ML50	16	0.95	0	23	0.0	0.55	1.3	0.43	10.37
CL95	120	0.45	62	15	4.0	1.00	21.2	0.13	20.42
CL90	75	0.54	48	17	7.0	0.94	10.2	0.17	19.64
CL85	50	0.60	41	18	8.0	0.90	5.2	0.21	18.85
CL80	35	0.66	34	19	8.5	0.87	3.5	0.25	17.60

Note: In the above table the soil type is defined as follows: SW = Gravelly sand, ML = Sandy silt, and CL = Silty clay. The compaction number is percent relative compaction, per AASHTO T-99. As an example, SW95 means gravelly sand compacted to 95% relative density per T-99.

Table 2.4: Duncan Soil Properties (Katona, 2015)

Soil Type and Compaction	Young's Tangent Modulus Parameters						Bulk Parameters		Density reference (kg/m3)
	K (--)	n (--)	c (Pa)	Φ_0 (deg)	$\Delta\Phi$ (deg)	R_f (--)	K_b (--)	m (--)	
CA105	600	0.40	0.00	42	9	0.7	175	0.2	23.57
CA95	300	0.40	0.00	36	5	0.7	75	0.2	21.99
CA90	200	0.40	0.00	33	3	0.7	50	0.2	21.21
SM100	600	0.25	0.00	36	8	0.7	450	0.0	21.21
SM90	300	0.25	0.00	32	4	0.7	250	0.0	19.64
SM85	150	0.25	0.00	30	2	0.7	150	0.0	18.85
SC100	400	0.60	3.44	33	0	0.7	200	0.5	21.21
SC90	150	0.60	2.07	33	0	0.7	75	0.5	19.64
SC85	100	0.60	1.38	33	0	0.7	50	0.5	18.85
CL100	150	0.45	2.76	30	0	0.7	140	0.2	21.21
CL90	90	0.45	1.38	30	0	0.7	80	0.2	19.64
CL85	60	0.45	6.90	30	0	0.7	50	0.2	18.85

Note: In the above table the soil type is defined as follows: CA = Coarse aggregate, SM = Silty sand, SC = Silty-clayey Sand, and CL = Silty Clay. The compaction number is percent relative compaction, per AASHTO T-99. As an example, SM100 means silty sand compacted to 100% relative density per T-99.

Table 2.5: Representative Material Properties (Katona, 2015)

Soil Type	Elastic Parameters, Nominal Range	
	Young's Modulus, E (kPa)	Poisson ratio, v (--)
Granular	4.1 to 13.8	0.30 to 0.35
Mixed	2.8 to 9.7	0.30 to 0.40
Cohesive	1.3 to 2.8	0.33 to 0.40

Note: Well-compacted soils are characterised by the high-range values of Young's modulus, whereas poorly compacted soils are characterized by low-range values.

2.2.2 Non Specified Materials

In the field, out of specification material is often utilised as backfill material when replacing existing culverts, for example utilising the material excavated from the trench. This is often because there is limited access to in spec material or cost savings are being sought and out of spec material is utilised. As Australian and AASHTO design standards rely upon the assumption of adequate backfill material it is also important to determine the potential risks of using out of spec material when this is not accounted for in the design. However, if the out of spec material is taken into consideration in the design, Committee:WS-006 (2007a) suggest reducing the bedding factor by 15% for material outside the standard grading. A bedding factor of 1.5 is taken where the fraction passing the 0.6 mm sieve is outside the limits (and not cement stabilised). The bedding factor is then applied to convert the load upon the pipe in-situ to a comparative test load by reducing the load, therefore a reduced bedding factor results in increased applied load. Cement stabilised material can be utilised as bedding and haunch zone material if outside these grading limits and this material may also be utilised as select fill for side zones if it meets the requirement for grading in AS/NZS 3725 (Committee:WS-006, 2007a). Cement stabilised sand is often the material utilised for both of these zones as it meets both requirements. However, there appears to be no control over mix design or strength gain within the standard and it is not mentioned as an acceptable material within the RMS specification (RMS, 2013). Transport and Main Roads Queensland (TMR, 2014) and Main Roads Western Australia (MRWA, 2013) both have clauses within their respective specifications that provide for the use of cement stabilised backfill (sand or other materials) both mentioning a mix proportion of 12:1 (sand to cement) when using an assumed uncompacted density of sand of 1200 kg/m^3. However, TMR do not specify this mix for culvert structures and it is noted in TMR (2013) to be excluded from use as a culvert backfill material due to adverse effects of moisture content of the soil.

Determining the effect of using materials outside of grading limits in a general sense can be achieved by determining the change in soil classification that would occur from deviating from specification. The use of high plasticity material for instance could result in a USCS graded CL (low plasticity inorganic clay) type material and utilising a material with oversize particles may result in a generally coarse aggregate with little fines (Katona, 2015). From this classification, standardised/approximate material properties can be identified within the literature in order to make reliable comparisons of the change in material. However, cement stabilised sand cannot be classified as a granular soil. The material properties for this material are largely affected by the properties of the constituents (sand, cement and admixtures) as well as its placement and compaction methods and curing (Morris & Crockford, 1991; Oliveria, Badelow, Wong, & Gorman, 2014). Few research papers were found that identify the material properties of stabilised sand when placed and compacted dry as a backfill material. However, from these papers general material properties could be identified to allow a comparison of this material to conforming backfill (Morris & Crockford, 1991; Oliveria et al., 2014). Material properties for out of specification granular materials are given in Table 2.2, with respect to the USCS grading types. The properties for stabilised sand for specific mix designs were identified by Morris and Crockford (1991) (Table 2.6). It was also noted that cohesion, elasticity, strength and internal friction increase as the cement content is increased.

Table 2.6: Stabilised Sand Properties (Morris & Crockford, 1991)

Cement Content (%)	Age (days)	Curing Humidity (%)	E (kPa)	c (Pa)	ϕ (deg)	f'c (kPa)
7	14	95	730.9	282	44	1586
7	75	95	875.7	490	42	2206
7	75	50	875.7	738	45	2206
5	7	95	262.0	179	39	827
5	21	95	441.3	269	35	1103
5	28	95	689.5	--	--	1172

2.2.3 Controlled Low Strength Material (CLSM)

CLSM is also outlined in AS/NZS 3725 (Committee:WS-006, 2007a) as a backfill material subject to certain conditions. These conditions include where;

- the project is subject to time and congestion restrictions,
- suitable compaction cannot be achieved due to minimal trench widths,
- spacing between adjacent pipes is less than recommended,
- the existing soil does not meet PSD conditions or stability requirements, and
- fill subsidence is required to be minimised.

CLSM is a soil cement slurry that sets into a stronger material than the surrounding soil. This material provides advantages by reducing costs associated with removing unsuitable material and reduced time and manpower associated with reduced compaction and testing requirements, which generally eliminates the possibility for rework to be required.

A CLSM requires the mix to be flowable. Where it does not meet slump requirements water should be added and it should be vibrated in order to improve the materials flowability. A CLSM also must have an unconfined compressive strength (UCS) within the range of 0.6 to 3.0

MPa and comply with mix proportions and material grading as outlined in Table 2.7 and Table 2.8 (Committee:WS-006, 2007a).

Table 2.7: CLSM mix proportions (Committee:WS-006, 2007b)

Material	% by weight	Standard
Portland cement	2-6	AS3972/NZS3122
Fly ash	0-20	AS3582.1
Granular soil material	60-80	AS/NZS 3725 Appendix A. A4
Water	10-20	AS1379

Notes:
1. Trial mixes should be prepared to confirm the strength characteristics and setting times of the selected mix, and to confirm mix suitability for the installation. It is important that the surrounding trench walls or embankment have a density and stiffness not less than that of the CLSM fill.
2. Mix strengths at the lower end of the range are usually re-excavatable.

Table 2.8: CLSM material grading (Committee:WS-006, 2007b)

Sieve Size (mm)	Percentage by weight passing (%)
19	100
0.075	0-25

There is no provision for CLSM within RMS specification R11 (RMS, 2013) however, TMR and MRWA both have provision for CLSM. These provisions are in the form of stabilised sand with sufficient water to ensure workability and compaction using concrete placement techniques or lean mix concrete (LMC) of 5 MPa strength and 40 mm nominal aggregate (MRWA, 2013; TMR, 2014). LMC does not meet Australian Standard specification given the target 28 day strength above the 3.0 MPa requirement and is not specified for culvert structures (Committee:WS-006, 2007a). MRWA (2013) suggest a ratio of 12:1 for cement stabilised backfill with sufficient water to allow compaction with an immersion vibrator. Material properties for a CLSM mix utilised for backfill were identified in Table 2.9 and Table 2.10.

Table 2.9: Backfill Properties for Controlled Low Strength Materials (Li-Jeng, Yeong-Nain, Darn-Horng, & Duc-Hien, 2014)

Dynamic Properties	E (GPa)	v	ρ (kg/m³)	G (μ) (GPa)	λ (GPa)	c1 = $\sqrt{\frac{\lambda+2\mu\rho}{\rho}}$ (Vc) (m/s)	c2 = $\sqrt{\mu\rho}$ (Vs) (m/s)
Soil	0.10	0.30	1745	0.0385	0.0577	277.75	148.46
CLSM (1 day)	0.12	0.25	2017*	0.0480	0.0480	267.20	154.27
CLSM (7 day)	0.14	0.25	1899*	0.0560	0.0560	297.44	171.72
CLSM (28 day)	0.47	0.25	1678*	0.1880	0.1880	579.75	334.72
CLSM (B80/30%)	0.27	0.25	1695*	0.1080	0.1080	437.21	252.42
CLSM (B130/30%)	0.87	0.25	1800*	0.3480	0.3480	761.58	439.70
Concrete	25	0.20	2322	10.4167	6.9444	3458.74	2118.04

Notes: * air dried

Table 2.10: Alternative Properties for Controlled Low Strength Materials (Zhan & Rajani, 1997)

Material	E (GPa)	v	ρ (kg/m³)	Φ (degrees)	Cohesion (kPa)
CLSM	0.20	0.30	2222	35	300

2.2.4 Open Graded Stone

An alternative backfilling method not mentioned in the Australian Standards or RMS specifications is the use of open graded stone or single sized aggregate. This material is considered self-compacting and reduces the requirement for testing. This method also eases placement difficulties around the haunches and between closely spaced structures (Tysl & Noll, 2011). The TGDSG (2014) drainage specification also allows the use of single sized aggregate for culvert bedding and support, however, this specification outlines the need to prevent the infiltration of water into the bedding. From initial assessment, if water infiltrates into the aggregate it may travel into the pavement structure and cause degradation, soil movement and settlement.

There has not been considerable research into the performance of this material when utilised as backfill, however, some standardised material properties were identified that would enable a comparison of this material when utilising suitable analysis techniques. These properties were identified by Gebrenegus, Nicks, and Adams (2015) and FHWA (2013) as shown in Table 2.11, Table 2.12 and Table 2.13. It is important to note that in both of these research papers the friction angle was determined from two separate approaches. The first approach is based upon the material meeting the requirements of a Mohr Coulomb soil failure envelope, however, this approach is limited by the assumption of cohesion within a cohesion-less material. The zero dilation angle (ZDA) approach utilises a linear relationship between the ZDA and the friction angle to approximate a friction angle based upon an assumption that the material is completely confined. This method does not suffer the same limitation of assuming cohesion within the material, however, it is considered to be potentially conservative (FHWA, 2013; Gebrenegus et al., 2015).

Table 2.11: Friction angles for Open Grade Stone (Gebrenegus et al., 2015)

AASHTO Gradation	Linear Mohr-Coulomb (MC)		Zero Dilation Angle (ZDA)
	ϕ'_t (degrees)	c-value (kPa)	ϕ'_{cv} (degrees)
Loose – No. 6	36.4	15.2	36.8
Dense – No. 6	40.8	10.3	36.9
Loose – No. 8	39.4	8.5	37.1
Dense – No. 8	40.3	22.8	38.8

Note: Friction angle was measured using two different approaches

Table 2.12: Characteristics of Open Graded Aggregate (Gebrenegus et al., 2015)

AASHTO Gradation	ρ_{min} (cm/g$_3$)	ρ_{max} (cm/g$_3$)	D_{max} (mm)	D_{50} (mm)	C_u	C_c
No. 6	1.61	1.77	25	12.9	1.79	1.07
No. 8	1.58	1.81	12.7	6.4	2.36	1.19

Table 2.13: Alternative Friction Angles for Open Grade Stone (FHWA, 2013)

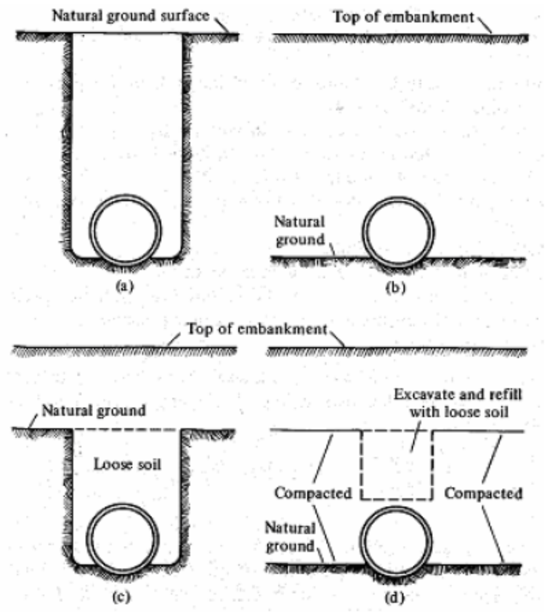
AASHTO Gradation	Friction Angle ϕ (degrees)			
	Mohr-Coulomb (MC)		Zero Dilation Angle (ZDA)	
	Dry	Saturated	Dry	Saturated
5	51	59	52	49
56	59	57	53	56
57	52	56	47	56
6	59	60	50	54
67	55	60	50	54
68	50	52	51	51
7	57	52	54	52
78	53	48	51	49
8A	54	50	52	50
8B	47	45	50	50
8B	47	45	50	50
8C	43	43	50	50
8D	52	46	53	50
9	53	45	52	48
10	46	41	46	44

The RMS have also identified open grade stone as an alternative backfill and bedding for around drainage structures and within trenches. Although not identified within the specification as a conforming method it has been utilised where trench conditions are excessively wet. This is to allow a freer draining path as well as reduce the difficulties associated with placing and compacting conforming backfill to the specification requirements in wet conditions. However, it is identified that certain procedures should be followed to ensure the material is properly compacted as standard compaction tests are not applicable for open grade stone (RTA, 2009).

It is also emphasised within RTA (2009) that the material can provide reduced support to pipe culvert walls, cause settlement and allow piping or erosion along the pipe or adjacent to the backfill. It is also not the preferred material for use in these conditions with geotextile and rock or no fines concrete being assessed as superior. It is also worth noting that this material requires a geotextile or other boundary layer to prevent the ingress of fines into the material which could lead to settlement adjacent to the pipe and cause reduced permeability of the trench (RTA, 2009).

2.3 Installation Conditions

Four conditions were specifically developed for the analysis of the loading of culvert structures. These conditions are the trench condition (or ditch condition), embankment installations which are either positive projection condition or negative projection condition, and the induced trench condition (or imperfect ditch condition) (Committee:WS-006, 2007a; Yoo et al., 2005). Figure 2.2 demonstrates each of these construction conditions. The trench condition is the case where a pipe is buried beneath the natural ground surface and frictional forces between the trench walls and the backfill provide support for the pipe. The embankment conditions occur when soil is placed above the natural ground level over the pipe. Those pipes that are partially or fully constructed above the natural ground surface are positive projections and those fully beneath the natural ground surface are negative projections. The induced trench condition occurs when an embankment is constructed over the pipe and an area of compressive material is placed above the pipe in order to reduce the effective load upon the pipe (Committee:WS-006, 2007a; Yoo et al., 2005).



(a) Ditch Conduit (b) Positive Projecting Conduit

(c) Negative Projecting Conduit (d) Imperfect Ditch Conduit

Figure 2.2: Installation Conditions (Yoo et al., 2005)

2.3.1 Trench Height

The height of backfill over a culvert structure has a direct impact on the loading of the culvert due not only to the potential change in dead load but also due to the changed distribution of soil stress. The change in dead load is directly related to the self-weight of the backfill that is supported by the culvert and with increased height there is increased soil load. Live load distribution also increases with trench height which causes a reduction in stress that results from the live load (Cook, Bloomquist, Gutz, & Ansley, 2002; NYSDOT, 2013). The way in which the live load is distributed by the soil can be analysed using several techniques. These techniques can include utilising approximate methods such as those provided in AS/NZS 3725 and AASHTO design standards, utilising linear elastic assumptions of the soil such as Boussinesq's theory or utilising non-linear soil models such as Duncan and Duncan Selig soil models (Committee:WS-006, 2007a; Cook et al., 2002; Das, 2010; NCHRP, 2010). These analysis techniques will be discussed in detail in the section on Analysis Methods below. Due to these relationships it can be seen there will be a height at which sufficient cover is maintained over the pipe to reduce the live load while minimising the cover to reduce the dead load upon the culvert.

2.3.2 Trench Width

The backfill loading that is supported by the pipe culvert is also a function of trench width. As the trench width increases there is an increase in pipe loading, assuming that the side fill is in a compacted state (NYSDOT, 2013; Yoo et al., 2005). This is due to the interaction of the backfill material with the sides of the trench. As differential settlement occurs the sides of the trench will resist the movement of the backfill material providing frictional support and reducing the loading upon the culvert (ConnDOT, 2000; NYSDOT, 2013).

2.4 Analysis Methods

The previously mentioned backfill types in combination with defined trench condition are utilised within the Australian Standard to determine the structural requirements of the pipe. However, Tran (2014) notes the potential for variations in practice when constructing the trench, laying the pipe and during backfilling. This variation can be assessed more accurately using the direct method by constructing a finite element pipe soil model for the calculation of loads, moments and shear along the pipe as identified by Erdogmus, Skourup, and Tadros (2010). The methods for the analysis of live loading upon the ground surface and dead loading from the soil was considered and compared against the AS/NZS 3725. It is also important to consider the potential for variation in results when certain assumptions about a soils response to loading are made. Often design methods and standards rely on linear elastic assumptions of a soil due to their ease of use, however, a soils response to loading is generally non-linear (Yoo et al., 2005).

2.4.1 Analysis of Live Loads

Consideration should initially be given to the method in which stresses within the soil profile are calculated. Using the fundamental principles of the mechanics of a deformable solid, Boussinesq developed a method for solving stresses in a homogenous, elastic and isotropic medium created from a surface point load in 1883, as shown in Equation 2.1 and Figure 2.3 (Boussinesq, 1883; Das, 2010).

$$\Delta\sigma_z = \frac{3P}{2\pi}\frac{z^3}{L^5} = \frac{3P}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}}$$

$$\Delta\sigma_x = \frac{P}{2\pi}\left\{\frac{3x^2z}{L^5} - (1-2\nu)\left[\frac{x^2-y^2}{Lr^2(L+z)} + \frac{y^2z}{L^3r^2}\right]\right\}$$

$$\Delta\sigma_y = \frac{P}{2\pi}\left\{\frac{3y^2z}{L^5} - (1-2\nu)\left[\frac{y^2-x^2}{Lr^2(L+z)} + \frac{x^2z}{L^3r^2}\right]\right\}$$

Equation 2.1: Boussinesq's Solution for a Point Load (Das, 2010)

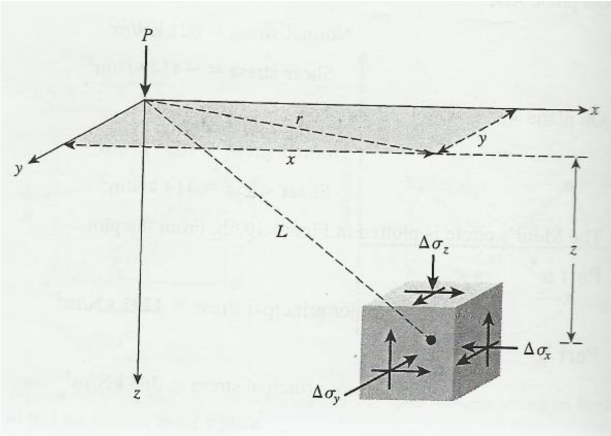


Figure 2.3: Stresses in an elastic medium caused by a point load (Das, 2010)

It is important to note that soil conditions are generally not ideal. The Westergaard method (developed in 1938) differs slightly from the Boussinesq approximation by considering the potential for alternating silt and clay layers, as shown in Equation 2.2. This was achieved by assuming the soil was an elastic medium interspersed with thin rigid layers that can only be displaced vertically. Despite the difference between the two methods the adapted method returns results quite similar to the Boussinesq method (Cook et al., 2002).

$$\Delta\sigma_z = \frac{P}{z^2\pi}\frac{1}{[1+2\left(\frac{r}{z}\right)^2]^{3/2}}$$

Equation 2.2: Westergaard Method (Cook et al., 2002)

At a similar time to Boussinesq's work Valentino Cerruti, an Italian mathematician, published a related paper that allows for the derivations of equations to calculate soil stresses given differing loading conditions (Cerruti, 1882; Kausel, 2010). These methods can consider vertical and horizontal line loads, vertical strip loads, embankment loading, circular loading, rectangular loading and other forms of surface loading (Cerruti, 1882; Das, 2010; Kausel, 2010).

For RCPC buried at depths greater than 0.4 m, the AS/NZS 3725 indirectly follows the Boussinesq method by expanding the footprint of the loading by 1.45 H (where H is the depth of fill). When less cover is available (depths less than 0.4 m), the standard considers the load to be acting directly upon the pipe but distributed over the effective length (Committee:WS-006, 2007a). AASHTO's method further reduces loading with depth by a factor of 1.75 H. This is due to the ability of pavements, designed for heavy traffic loading, to distribute loading throughout the base and sub-base layers (Sezen, Fox, & Yeau, 2009).

2.4.2 Analysis of Dead Loads

Marston and Spangler developed the initial concepts for bedding conditions and loading on buried conduits in the early 20th century. Their work involved solving a series of differential equations based upon construction and installation methods to determine design coefficients to calculate the soil loads (Marston, 1930; Spangler, 1951; Yoo et al., 2005). The method varies depending upon the four main types of installation condition identified previously as the trench condition, embankment conditions (positive projection and negative projection) and the induced trench condition. Based upon these conditions, as well as the backfill type and trench width, Marston and Spangler identified the loading upon the culvert by applying a bedding factor to the unit weight of culvert above the pipe (Erdogmus et al., 2010; Kim & Yoo, 2005; Yoo et al., 2005). This bedding factor was based upon the width of the trench, the contact between the pipe and bedding, the magnitude of lateral earth pressures and the height over which it acts (Loo & Chowdhury, 2010). Li and Aubertin (2014) identified that Marston and Spangler's initial method can often lead to underestimation of soil stress in the lower part of the trench or overestimation of soil stresses when constructing a trench with sloping walls. Erdogmus et al. (2010) also indicated that the development of soil theories and finite element programs has identified potential inaccuracies of Marston and Spangler's initial theories. Handy (2004), in his paper *Anatomy of an Error*, discussed some potential inconsistencies with Marston and Spangler's works including inability for a ratio of principle stresses (horizontal to vertical), denoted as K, to develop friction. He also suggested that the Rankine earth pressure coefficient is not equivalent to this ratio of stresses as Marston stated. It is also recognised that early definitions of compaction were lacking and could not be accurately identified in Marston and Spangler's testing (Yoo et al., 2005).

The AASHTO standards for culvert installation further developed upon Marston and Spangler's work by utilising improved Finite Element Method (FEM) analysis techniques. In particular, the computer program Soil Pipe Interaction Design and Analysis (SPIDA) was utilised for the direct design of buried pipes. This analysis significantly improves upon Marston and Spangler's methods by adequately considering lateral and axial forces and recognising the limitations of test methods utilised by Marston and Spangler (Yoo et al., 2005). The AASHTO method utilised the worst case of positive projection embankment design with the direct design method, utilising SPIDA, and developed an indirect design based upon these tests. AS/NZS 3725 also provides a similar indirect method which has been developed from Spangler's early theories, American Society for Testing and Materials standards, direct design testing utilising SPIDA, research from the Adelaide University as well as research from the American Concrete Pipe Association (Committee:WS-006, 2007b). From this, bedding factors and loading upon pipes could be developed. The ease of use of these indirect methods has led to their success in industry (Tran, 2014).

2.5 Soil Models

The development of finite element programs for pipe design and installation have been improved and studied since the 1970's (Abolmaali & Kararam, 2013). This method of analysis allows for improvements upon earlier approximations that were limited by certain assumptions such as soil being linear, elastic and homogenous (Duane, Robinson, & Moore, 1986). This method does not just consider the worst case but takes into account numerical approximations of partial differential equations along a finite series of soil elements. SPIDA (as mentioned earlier), Culvert Analysis and Design (CANDE), PIPE5 (developed from SSTIPN) and PLAXIS are all examples of geotechnical finite element analysis software that may be utilised for the determination of pipe loading using a non-linear soil model (Abolmaali & Kararam, 2013; Aldous, 2008; Crosby, 2003).

Due to the complexity and the excessive calculations required to analyse a soil profile of varying properties and response, approximate methods have been readily adopted. Now that analysis can be easily undertaken directly utilising a soil behaviour model, there is potential to optimise design and construction procedures. However, this can only be achieved by understanding the variations and strengths of particular soil models.

2.5.1 Mohr-Coulomb Soil Model

Mohr-Coulomb's theory determines that a granular material will have an initial linear elastic response to loading, however, upon failure the soils behaviour is plastic. This theory states that a material will fail due to a critical combination of normal stress and shearing stress and presents a linear relationship between these properties (Equation 2.3) (Das, 2010) . Using the Mohr-Coulomb Model to represent live load distributions around buried structures allows for adequate analysis of the live load distribution to structures while also remaining computationally simplistic (NCHRP, 2010). Disadvantages of this model, due to the assumption of linear elasticity of the soil, arise from the way soil failure is instant rather than gradual and there is no stiffening of the soil modulus as confining pressures increase (Katona, 2015). However, it is important to note that this model does represent stiff clays well.

The Mohr-Coulomb failure occurs due to the maximum achievable shear stress as defined by the relationship of normal stress to material properties for cohesion and internal friction. This is shown in Equation 2.3 and can be represented as shown in Figure 2.4 by showing the variation of the failure surface against shear stress and normal stress.

$$\tau_{max} = c + \sigma_n tan\Phi$$

τ_{max} = Maximum Achievable Shear Stress (Shear stress at failure)

c = Cohesion

σ_n = Normal stress on the plane of failure

ϕ = Angle of internal friction

Equation 2.3: Mohr Coulomb Shear Stress (Katona, 2015)

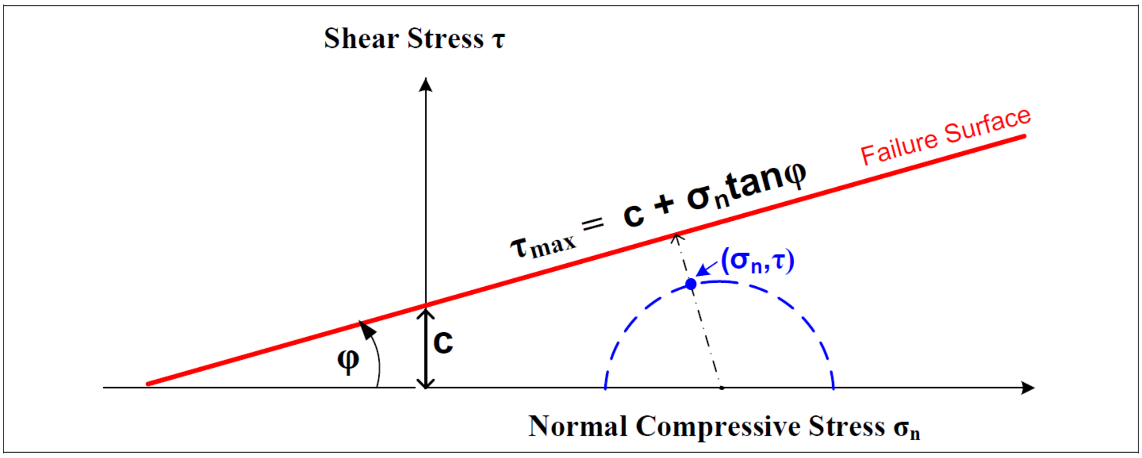


Figure 2.4: Mohr-Coulomb Failure Surface (Katona, 2015)

2.5.2 Duncan and Duncan Selig Soil Model

Hyperbolic soil models are often utilised due to their ability to represent non-linear soil relationships (Yoo et al., 2005). A disadvantage of this type of model is that it represents the soil failure as abrupt rather than gradual as is the case with backfills (Katona, 2015). The Duncan soil model approximates the Elastic Modulus and Bulk Modulus using Equation 2.4 and Equation 2.5. Selig improved upon the model by incorporating a more realistic estimation of the Bulk Modulus through Equation 2.6. This incorporation resulted in the Duncan Selig model. Katona (2015), through the development of the CANDE program, also introduced a modification to allow the simulation of plastic behaviour and deformation of the soil upon unloading.

$$E_t = E_i \left[1 - \frac{R_f(1 - sin\Phi)(\sigma_1 - \sigma_3)}{2(Ccos\Phi + \sigma_3 sin\Phi)} \right]^2$$

E_t = Tangents Young's Modulus dependent upon stress state

E_i = Initial Young's Modulus (at zero stress)

R_f = Duncan Failure Ratio

σ_1 = Maximum principal compressive stress

σ_3 = Minimum principal compressive stress

Equation 2.4: Young's Modulus variation with stress - Duncan Model (Katona, 2015)

$$B_t = K_b P_a (\frac{\sigma_3}{P_a})^m$$

Where;

$$K_b = Dimensionless\ magnitude\ of\ tangent\ bulk\ modulus$$

$$P_a = Atmospheric\ pressure$$

$$m = Power\ function$$

$$\sigma_3 = Confining\ pressure$$

Equation 2.5: Bulk Modulus variation with stress - Duncan Model (Katona, 2015)

$$B_{t\ (s)} = B_i\ [1 + \frac{\sigma_m}{B_i \varepsilon_u}]^2$$

$B_{t\ (s)}$ = Selig tangent bulk modulus

B_i = Initial tangent bulk modulus when volumetric strain = 0

ε_u = Ultimate volumetric strain at large hydrostatic stress

σ_m = Mean Stress

Equation 2.6: Bulk Modulus variation with stress - Duncan-Selig Model (Katona, 2015)

This research has allowed for analysis of soil structure interactions with a stress dependant response to loading and plastic deformation following failure (Katona, 2015; NCHRP, 2010).

2.5.3 Hardin Soil Model

The Hardin soil model is based upon a hyperbolic relationship between shear stress and shear strain which increases stiffening of the constitutive modulus when confining stress increases and softening when shear stress increases. This model is similar to the Duncan and Duncan

Selig soil models, however, it is not as popular as the later Duncan and Duncan Selig models. This is due to the reduced soil parameters considered, although the Hardin model does characterise the shear modulus parameters in terms of fundamental soil properties (void ratio, PI and percent saturation) (Katona, 2015). Like the Duncan and Duncan Selig Model the Hardin soil model also has a variable modulus of elasticity, shear modulus and Poisson's ratio dependant on stress strain relationships (Katona, 2015).

2.6 Failure of Culverts

The failure of pipe culvert structures can occur in a number of manners due to a range of contributing factors. The main factors that contribute to a culverts failure include:

- incorrect construction methods,
- incorrect design/incorrect assumptions,
- changes to the conditions assumed within the design (introduction of heavy vehicle/axle loadings),
- erosion of backfill or bedding material,
- lack of maintenance, and
- corrosion (Beaver et al., 2004; Tan & Moore, 2007; Wagener & CNAConsultingEngineers, 2014).

The incorrect construction of culverts can be caused by changes to the backfill material used onsite, insufficient backfill compaction, poor foundation material, incorrect jointing and improper laying of the pipes (Rajah et al., 2012). This can have a range of impacts including increased culvert loading, migration of bedding and backfill material, settlement of the pavement and structural damage to the pipe (Yoo et al., 2005). Incorrect design and implementation should not be a contributing factor to culvert failure and in most cases the issue arises where there is a change from the initial design considerations (potentially years down the track) or construction practices are not consistent with design assumptions. Many cases of culvert failure relate to the formation of voids in the culvert bedding and backfill material. This is often associated with cracks or disjointing in the culvert that can lead to water infiltration into the bedding and backfill and a migration of fine particles. The void formation can then lead to an increase in moments about the pipe as well as movement of the pipe and potentially further joint separation (Tan & Moore, 2007). The movement of material may significantly affect the structural capacity of the pipe and lead to deterioration and collapse (Wagener & CNAConsultingEngineers, 2014). Where such failures occur and there is potential erosion of material or pavement failure above the pipe, structured remedial maintenance programs can often improve the lifespan of a deteriorated culvert. This is also the case for identifying potential corrosion issues due to deterioration of the pipe from potentially aggressive conditions. If the culvert is assessed early and there is potentially exposed reinforcement or concrete deterioration remedial action can take place prior to excessive damage of the culvert.

2.7 Conclusions and Future Research

The culvert design and analysis process has been a developing area since the early 20th century with various analytical, empirical and numerical methods utilised. The current design standards analysed in this literature review retain the use of indirect empirical methods even with the development that has occurred in finite element analysis methods since the 1970s. These empirical methods are still used even when considering the advancement in soil models that represent actual soil behaviour by utilising finite element software packages to iteratively evaluate a soils response to loading and failure over a range of conditions. Over the period of research there have also been changes in construction methods with many different culvert construction and backfill methods being utilised in industry. These changes were often developed due to either poor implementation of design and quality control, a lack of available materials and potential construction constraints or due to time restrictions. Based upon this review it can be seen that further research analysing current design standards and optimising design methods is necessary. An understanding of the effects of design standard limitations and construction methods will assist in improving the safety of design and implementation of RCPC. Conveying these results in the implementation stages of projects is also important in order to achieve satisfactory design outcomes.

3 Project Methodology

3.1 Outline

The project has been broken up into the analysis of the effect of several factors on backfill displacements, stresses and strains and the resulting load upon the pipe. These factors include the installation condition, wheel load, trench width, pipe diameter, backfill type and analysis method. The design criteria to be analysed has been summarised and presented in Table 3.1 for simplicity.

Table 3.1: Comparison Table (Committee:WS-006, 2007a; Das, 2010; Gebrenegus et al., 2015; Katona, 2015; Li-Jeng et al., 2014; Morris & Crockford, 1991; Rajah et al., 2012; Yoo et al., 2005)

Installation Condition	Load	Trench Width (mm)	Trench Depth (mm)	Pipe Diameter (mm)	Backfill Type	Analysis Method
Trench Condition (vertical walls)	W80	100*	300	600	BH (SM100 & SW100)	Linear Elastic Assumptions
		150	600		SO (SM100, SW100 & SC100)	Mohr-Coulomb Model
			1200		Out of spec gravel (CA105, CL95 & CL100)	Duncan Model
					Open Graded Stone (No. 6 and No. 8)	AS/NZS 3725
					CLSM (1 day, 7 day & 28 day)	
					Stabilised Sand	

Note: *CLSM only (Committee:WS-006, 2007a)

3.2 Modelling

3.2.1 Model Types

The project will be undertaken using existing simplified analysis methods which rely on linear elastic and homogenous assumptions for the soil medium with each design scenario. The soil models will be utilised from the software package CANDE. This program was originally released in 1976 under sponsorship of the Federal Highway Administration (FHWA) (Katona, 2015). Since then the program has undergone development to improve analysis techniques, useability and allow for further design applicability harnessing improved computer processing capabilities. CANDE was selected due to its wide range of available soil models, availability of source coding and user documentation as well as being free to use.

Within this program the soil model, trench conditions, culvert material type and the interface between the structure and the backfill can be defined in order to determine the stresses within the culvert backfill and transferred to the culvert structure. The programs Graphical User Interface (GUI) can be utilised for direct input and analysis of these conditions. However, the updated features that allow for non-linear analysis of soils using the Duncan and Duncan Selig models as well as the Mohr Coulomb model have not been incorporated into the GUI (Katona, 2015). Therefore these conditions require input via direct text input.

The models selected for use within the project were selected due to their applicability to the culvert design problem. Simplified linear elastic methods were directly compared to the more complex and potentially more representative soil models. The Mohr Coulomb model was selected for comparison due to its general acceptance by geotechnical engineers to approximate shear failures and for analysing soil structure interactions (Das, 2010; Katona, 2015). This model is also accepted and utilised by commercial analysis programs such as ABAQUS, NASTRAN and FLAX3D. The final soil model for comparison was the Duncan soil model due to the ability to represent actual soil behaviour and the Duncan and Duncan Selig models are described as effective for representing the stress dependant behaviour of culvert backfill (Katona, 2015). It is also important to note that the in-situ material was modelled as linear elastic for all model types. The in-situ material is likely to be well compacted, have settled post construction and would be generally homogenous hence the model choice. This is also consistent with the modelling of Kitane and McGrath (2006) as well as CNAConsultingEngineers, Simpson, Gumpertz, and Heger (2009).

3.2.2 Culvert Scenarios

The chosen design scenarios were developed based upon current construction practices and design assessment methods within the Australian Standards as well as historical development of culvert analysis techniques originating with Marston and Spangler. The current standard AS/NZS 3725 (Committee:WS-006, 2007a) utilises the installation conditions originally developed by Marston and Spangler and the condition selected for assessment within this project is the trench condition. AS/NZS 3725 identified different analysis methods for varying backfill heights and changed trench widths. This is why backfill heights less than 300 mm, between 300 and 1000 mm and greater than 1000 mm have been selected for comparison (Committee:WS-006, 2007a; Yoo et al., 2005). However, the variation in width was restricted by CANDE due to inbuilt criteria for trench width (the width is required to be between 1.25 and 1.5 times pipe diameter) (Katona, 2015). The change in analysis method with changing backfill height has been simplified within AS/NZS 3725. If the height of the backfill is less than 0.4 m in height then the wheel load is considered to act directly on the top of the pipe. Where the height is greater than 0.4 m a wheel load is to be distributed over an area that increases by 1.45 times with increased depth. However a range greater than 1 m was also included in the analysis to compare the variation between models and standard methods over deeper trench depths. The widths chosen have been incremental increases from the minimum of 150 mm (for granular backfill) and 100 mm (for CLSM) from the edge of the pipe to the sides of the trench. The pipe diameter of 600 mm was chosen for simplicity since it represents a typical size for pipe culverts.

The loading condition was also selected based upon the current Australian Standard for bridge loading, AS 5100.2, to allow a comparison between the effects of loading conditions for each of the model types based upon theoretically acceptable standard loading (Committee:BD-090, 2004). The standard loadings given in AS 5100.2 under SM1600 loading have been refined to only include the W80 wheel load. M1600 and S1600 represent moving and static loadings separated over a large distance and the A160 load is a single axle load. These have not been included in the modelling. The separation between these loads means that any added loading from several loads is negated due to the stress reduction through the pavement material surrounding the culvert structure. These load conditions are also restricted due to the limitations of having two axles which would alter the way in which the live loading was converted for use in CANDE (Committee:BD-090, 2004).

3.2.3 Backfill Types

Selected backfill types to be compared were based upon industry experience and construction practices as well as acceptable methods utilised within different state government legislation and Australian Standards. These standards and specifications included certain criteria for each conforming granular backfill type which were then compared to grading requirements within the USCS and current literature to determine the material properties. CLSM and stabilised sand also had identified requirements within the standards and specifications, which enabled research to identify currently acceptable ranges of material properties. There was some difficulty in identifying information surrounding open graded stone utilised as a backfill material due to limited documentation on its use. However, through the use of state highway authorities specifications in conjunction with literature on aggregate materials the approximate material properties to be utilised for analysis could be identified.

The granular backfill types were identified by the USCS as SM100 and SW100 for bedding and haunch zones and SM100, SW100 and SC100 for side and overlay zones (Committee:WS-006, 2007a; Katona, 2015; Rajah et al., 2012; RMS, 2013). For granular material that was to be considered as out of specification, higher clay contents and increased oversize material were considered the most common issues. Therefore, in order to identify the material properties consistent with the literature review, the USCS classifications of CA105, CL95 and CL100 were given (Katona, 2015; Rajah et al., 2012). Material properties for stabilised sand and CLSM were adapted from literature on field testing. The mixes utilised within these experiments varied from mix designs identified within the standards as well as state highway specifications. The basic material properties could be adjusted through the identification of the relationships between the properties and mix design.

The standards and specifications adopt a requirement for 12:1 soil/sand to cement ratios (approximately 8%) for both CLSM and stabilised sand (Committee:WS-006, 2007a; Li-Jeng et al., 2014; Morris & Crockford, 1991; MRWA, 2013; TMR, 2014). As mentioned, the open graded stone had limited literature surrounding its use as a backfill material. However, it was identified that such materials would correspond approximately with AASHTO graded No.6 and No. 8 gravels, which were considered to be equivalent to a 14 and 7 mm aggregate, respectively, for the purposes of this research. From this grading, certain material properties which would allow for direct analysis using the planned soil models could be identified from the literature (FHWA, 2013; Gebrenegus et al., 2015).

The material properties selected for the analysis of each backfill material, based upon the existing research, is shown in the following tables (Table 3.2 and Table 3.3).

Table 3.2: Backfill Material Properties (FHWA, 2013; Gebrenegus et al., 2015; Katona, 2015; Li-Jeng et al., 2014; Morris & Crockford, 1991; Rajah et al., 2012; Zhan & Rajani, 1997)

		Elastic		Mohr-Coulumb		
		Young's Modulus	Poissons Ratio	Cohesion	Internal Friction Angle	Unit Weight
Backfill Type		E (MPa)	v	c (kPa)	(ϕ)	γ (kN/m3)
BH Type	SM100	8.27	0.33	0.00	36.00	18.86
	SW100	11.03	0.33	0.00	36.00	19.64
SO Type	SM100	8.27	0.35	19.15	33.00	18.86
	SW100	11.03	0.33	0.00	36.00	19.64
	SC100	8.27	0.35	19.15	33.00	18.86
Oversize	CA105	11.72	0.33	0.00	36.00	19.64
High PI	CL95	5.86	0.35	11.97	30.00	17.76
	CL100	6.89	0.35	14.36	30.00	18.07
Open Graded Stone	No6	20.68	0.33	10.30	40.80	17.36
	No8	20.68	0.33	22.80	40.30	17.76
CLSM	1day	120.00	0.25	NA	NA	19.79
	7day	140.00	0.25	NA	NA	18.63
	28day	470.00	0.25	NA	NA	16.46
CLSM Alternative		200.00	0.30	300.00	35.00	21.80
Stabilised Sand		875.00	0.25	0.74	44.00	23.57

Table 3.3: Duncan Selig/Duncan Soil Model Backfill Material Properties (Katona, 2015)

Duncan Selig or Duncan Model parameters								
Backfill Type	K	n	c (kPa)	ϕ	Δϕ	R_f	B_i/P_a or Kb	ε_u or m
SM100	600.00	0.25	0.00	36.00	8.00	0.70	450.00	0.00
SW100	1300.00	0.90	0.00	54.00	15.00	0.65	108.80	0.01
SM100	600.00	0.25	0.00	36.00	8.00	0.70	450.00	0.00
SW100	1300.00	0.90	0.00	54.00	15.00	0.65	108.80	0.01
SC100	400.00	0.60	3.45	33.00	0.00	0.70	200.00	0.50
CA105	600.00	0.40	0.00	42.00	9.00	0.70	175.00	0.20
CL95	120.00	0.45	62.05	15.00	4.00	1.00	21.20	0.13
CL100	150.00	0.45	2.76	30.00	0.00	0.70	140.00	0.20

3.3 Inventory Data

The inventory data for two sites were collected in conjunction with the RMS asset maintenance section and analysed utilising the CANDE program and the same procedures outlined for the design scenarios. The information surrounding these culverts was collated from field investigations and the culvert structure was analysed utilising the same methods utilised for comparing the design scenarios. Two culverts were identified for assessment during this process.

The following sections provide the details of the two case studies. These case studies were undertaken on culverts that utilised alternative backfilling techniques (outside of those identified within RMS specification R11 and the AS/NZS 3725). Although, it is important to note that they are not alternative in the fact that AS/NZS 3725 does not restrict alternative backfill rather, it provides additional safety factors/reduction factors when designing the culvert. These case studies were both from culverts constructed within the RMS Northern region in NSW.

3.3.1 Culvert Case 1

The first case study that was analysed was a 750 mm class IV pipe culvert. This culvert was backfilled utilising a single sized aggregate backfill (14 mm aggregate) for which the material properties of the No6 graded gravel were selected. This pipe had a cover of 400 mm, at the lowest point, below the pavement and a trench width of approximately 1050 mm.

3.3.2 Culvert Case 2

The second case study that was analysed was a 600 mm class IV pipe culvert. This culvert was backfilled with conforming granular backfill initially. However, due to the age of the pipe and lack of cover failure occurred and in order to account for the low cover of 300 mm the pipe was backfilled with stabilised sand. This pipe had a cover of 300 mm, at the lowest point, below the pavement and a trench width of approximately 900 mm.

3.4 AS/NZS 3725 Analysis

Each of the selected design scenarios, backfill types and the case studies were also analysed utilising the Australian Standard Design for Installation of Buried Concrete Pipes (Committee:WS-006, 2007a). This was undertaken as a check to see the consistency between the modelling and the standard. The standard provided a comparison of failure only.

The standards design process involved determining an installation condition, construction dimensions and materials to be utilised for the culvert installation. The design scenarios outlined in Section 3.2.3 of this report were utilised for the comparison. From this information, the dead load, in force per unit length, upon the pipe can be calculated for the trench condition by Equation 3.1.

$$W_g = C_t w B^2$$

$$C_t = Coefficient\ for\ trench\ installation$$

$$w = Unit\ weight\ of\ backfill\ material\ (\frac{kN}{m^3})$$

$$B = Width\ of\ trench\ (measured\ at\ the\ top\ of\ the\ pipe)$$

Equation 3.1: AS/NZS 3725 Dead load for trench condition - Section 6.3.2 Equation 1 (Committee:WS-006, 2007a)

The live load, in force per unit length, is then determined in clause 6.5.3.2 of the Australian Standard (Committee:WS-006, 2007a) based upon the identified live loading and the determination of the loaded area beneath the backfill. This is undertaken utilising two different methods. If the cover to the pipe is less than 0.4 m then the load is transferred directly to the pipe. For cover depths greater than 0.4 m the load is distributed over a load area that is linearly increased with depth (Equation 3.2).

$$A = (b + 1.45H)(a + 1.45H)$$

$$a = Length\ of\ tyre\ contact\ area$$

$$b = Width\ of\ tyre\ contact\ area$$

$$H = Fill\ height\ over\ the\ pipe$$

Equation 3.2: Load distribution – Section 6.5.3.2.1 (Committee:WS-006, 2007a)

The live load is then calculated using Equation 3.3 and Equation 3.4.

$$q = (1 + \alpha)(\Sigma P)/A$$

$$\alpha = Dynamic\ load\ allowance\ (0.4)$$

$$P = Wheel\ load$$

Equation 3.3: Average Live Load – Section 6.5.3.2.1 (Committee:WS-006, 2007a)

$$W_q = qL_1S/L_e$$

$$S = Lesser\ of\ L_2\ or\ S$$

$$L_2\ is\ determined\ in\ accordance\ with\ Figure\ 9\ of\ AS3725$$

$$L_1\ and\ L_e\ are\ determined\ in\ accordance\ with\ figure\ 10\ of\ AS3725$$

Equation 3.4: Working Live Load - Section 6.5.3.4.1 (Committee:WS-006, 2007a)

The total proof load can then be calculated by applying specific bedding factors to both the live and dead load based upon the material properties and compaction conditions. The final proof load is calculated in accordance with Equation 3.5.

$$T_c = \frac{W_g}{F} + \frac{\Sigma W_q}{F_q}$$

$$F = Bedding\ factor\ determined\ in\ accordanc\ with\ Clause\ 6.5$$

$$F_q = The\ lesser\ of\ 1.5\ and\ F$$

Equation 3.5: Proof Load - Section 10.2 (Committee:WS-006, 2007a)

4 Introduction to CANDE

4.1 Outline

CANDE is a finite element program developed specifically for analysis of the culvert soil interaction and to improve the structural design of culvert structures. This program was first released in 1976 under sponsorship from the FHWA of the United States of America (USA). The development of this program has continued over the years under the sponsorship of organisations, such as the AASHTO, the Transportation Research Board (TRB) and private industry, which has resulted in the release of the latest version of CANDE in 2015. This version of the program offers a range of improvements over previous versions due to the development of technology and advancements in geotechnical engineering. This has enabled the development of improved soil models that can better simulate soils behaviour.

CANDE allows for a range of analysis methods including assuming an elastic, homogenous and infinite soil backfill and two finite element solutions made up of discrete soil layers with one mesh automatically generated and the other defined by the user. Interface elements between the pipe and soil can be defined by each of these solution methods, with the FEM allowing for separation and re-bonding between the interface, a fully bonded interface and a frictionless interface that only transmits normal forces (Katona, 2015). However, for the purposes of this project, interface elements have not been considered as it was not directly required to meet the aims of this project and it was found in literature to have an insignificant difference upon results (Kim & Yoo, 2005). The elements employed by CANDE's FEM mesh include:

- Quadrilateral and triangular elements for the in-situ soil and backfill material,
- Interface elements for the pipe soil connection,
- Link elements for nodal connections, and
- Beam-Column elements for structural elements (culvert pipes, box culverts, arches, etc.).

The FEM model is created in distinct construction increments to facilitate the calculation of nodal forces as the culvert is backfilled and live loading is applied. The FEM model is two dimensional and therefore the properties and loading cannot be varied in the Z direction (along the pipe). The main impact of this is for the application of live loading. A tyre or axle load is applied as a uniform load over a particular area and this load must be modified in order to match the stress distribution created by an infinite strip load. This means that the distribution of the live load in the Z direction must be approximated with simpler methods (as opposed to the built-in soil models applied by CANDE). This involves utilising the approximations created by standards or determining an equivalent strip pressure from linear elastic methods such as those derived by Boussinesq. For this project Boussinesq's equations for the change in pressure with depth from an infinite strip width and rectangular loaded area were utilised to determine the equivalent pressure to be applied in CANDE. This was achieved using Equation 4.1, with reference to Figure 4.1, Equation 4.2 and Equation 4.3.

$$\Delta\sigma_{zstrip} = \frac{q_{strip}}{\pi}\left[\alpha + \sin(\alpha)\cos(\alpha + 2\beta)\right]$$

Where;

$$\beta = tan^{-1}\left(\frac{x - \frac{B}{2}}{z}\right)$$

$$\alpha + \beta = tan^{-1}\left(\frac{x + \frac{B}{2}}{z}\right)$$

Equation 4.1: Induced vertical stress caused by a uniform strip load (Ghabraie, n.d)

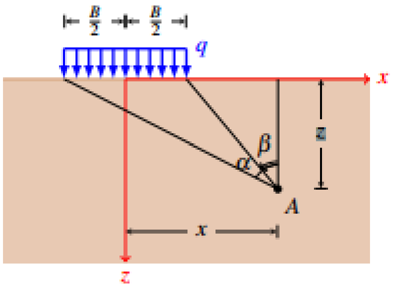


Figure 4.1: Vertical stress caused by a uniform strip load (Ghabraie, n.d)

$$\Delta\sigma_{zpatch} = q_{patch}I_4$$

Where;

$$I_4 = \frac{2}{\pi}\left[\frac{m1n1}{\sqrt{1 + m1^2 + n1^2}}\frac{1 + m1^2 + n1^2}{(1 + n1^2)(m1^2 + n1^2)} + sin^{-1}\left(\frac{m1}{\sqrt{m1^2 + n1^2}\sqrt{1 + n1^2}}\right)\right]$$

$$m1 = \frac{L}{B}, (L > B)$$

$$n1 = \frac{2z}{B}$$

Equation 4.2: Induced vertical stress caused by a uniform patch load (at the centre of the patch) (Ghabraie, n.d)

When $\Delta\sigma_{zpatch} = \Delta\sigma_{zstrip}$

$$q_{strip} = q_{patch} I_4 / \left(\frac{1}{\pi}\left[\alpha + \sin(\alpha)\cos(\alpha + 2\beta)\right]\right)$$

Equation 4.3: Equivalent strip pressure for use with CANDE

The pressures to be applied in CANDE caused by an 80 kN W80 wheel load directly above the centre of the pipe are shown in Table 4.1.

Table 4.1: CANDE Live Load Input

Analysis Type	0.3 m (kN/m)	0.6 m (kN/m)	1.2 m (kN/m)
Boussinesq	92.1	57.0	30.5
AS/NZS 3725	200.0	44.6	50.2*
AASHTO	84.0	53.2	30.7

*AS/NZS 3725 at 1.2 m depth considers interaction from both wheels on a single axle

From the above data, the Boussinesq method was utilised for comparison within the models as it represents a worst case scenario (in most cases) and is not as simplified as the AS/NZS 3725 and AASHTO methods. The AS/NZS 3725 method was not selected as the cases where it exceeds the Boussinesq method are due to approximations that may not occur in reality. For example the 300 mm cover depth takes the load directly on the pipe rather than distributed through the 300 mm cover depth. The 1200 mm cover is only greater than the Boussinesq method as an overlap is assumed between the two wheel loads which would also be unlikely in reality.

The required input parameters for culvert analysis are summarised as follows:

- Culvert type (RCPC, high density polyethylene, corrugated metal, etc.)
- Solution type (elasticity based, auto FEM mesh, user defined FEM mesh)
- Pipe soil interface type
- Trench condition (trench, embankment or homogenous)
- Soil model
- Culvert material properties (elasticity, steel reinforcement, Poisson's ratio, compressive strength, etc.)
- Culvert and trench geometry (pipe diameter, culvert height, culvert width, trench depth, trench width)
- Number of load steps
- Live loading
- Soil properties (density, elasticity, Poisson's ratio, angle of internal friction, etc.)

The input parameters to be compared were identified within the Project Methodology section of this report. It is important to note that some input parameters have programmed restrictions which limit the possible value. For example the trench width is limited to 1.2 to 1.5 times the culvert diameter. It is for this reason that trench width has been removed as a parameter to be compared within this project (except for the retained 100 mm width for CLSM). Alternative inputs required by CANDE are shown in Table 4.2.

Table 4.2: Input Parameters

Input	Selection
Type of Analysis	Analysis
Method of Analysis	Service
Solution Level	FEM auto mesh (Elasticity Level 2)
Interface Elements	None
Compressive Strength	60 MPa
Young's Modulus (Pipe)	34,800 MPa
Poisson's Ratio (Pipe)	0.17
Unit Weight of Concrete	24.3 kN/m^3
Crack Width Model	Heger-McGrath
Yield Stress of Reinforcing Steel	500 MPa
Young's Modulus of Reinforcing Steel	200 GPa
Poisson's Ratio of Reinforcing Steel	0.3
Steel Reinforcement Spacing	50.8 mm (outside layer only)
Steel Type	Smooth wire or plain bars
Non-Linear Behaviour	Plus steel yielding and plastic behaviour
Steel Area/Cover	0.635mm^2/mm / 20 mm
Bedding Depth	150 mm
Pipe Internal Diameter*	586 mm
Pipe External Diameter*	698 mm

*Standard Class 4 Humes 600 mm pipe (Humes, 2009)

The automatically generated mesh required slight adjustments in order to make it specific to the design cases. Seven nodes were required to be moved in order to ensure a bedding depth of 150 mm beneath the pipe (Table 4.3). The live load was also applied to two surface nodes over an approximate spacing of 100 mm (Table 4.4) and over 5 construction increments to limit the impacts on convergence.

Table 4.3: Node Adjustment

Node	X coordinate	Y coordinate
22	0	18.6615
23	6.56846	18.6615
24	14.8447	18.6615
25	25.2728	18.6615
26	38.4122	18.6615
27	54.9678	18.6615
28	75.828	18.6615

Table 4.4: Live load

Node	0.3 m (kN/m)	0.6 m (kN/m)	0.9 m (kN/m)	1.2 m (kN/m)
103 / 104	4.60	2.85	2.00	1.52

The interface elements were not utilised within this project. This option has been assessed and identified as creating an insignificant impact upon the results. This was also identified by Kim and Yoo (2005).

The parameters are utilised for each of the following analysis cases shown in Figure 4.2, Figure 4.3 and Figure 4.4.

Note:

- These cases are the same for the 100 mm CLSM trench width, except with a reduced trench width and therefore slightly altered FEM mesh.
- The live load is half the load calculated in Table 4.1 due to the requirement to split the load over two elements.

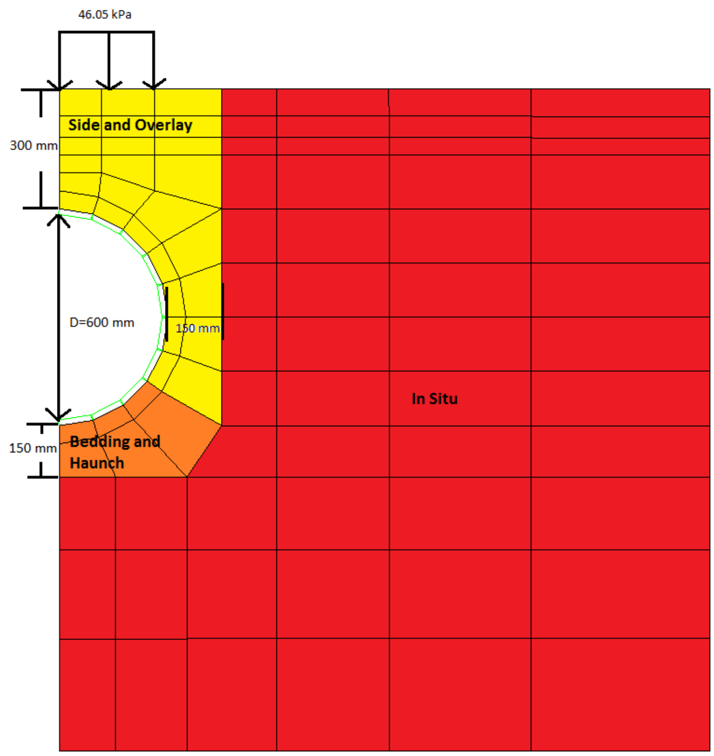


Figure 4.2: Finite Element Mesh for Design Scenarios 300 mm Cover

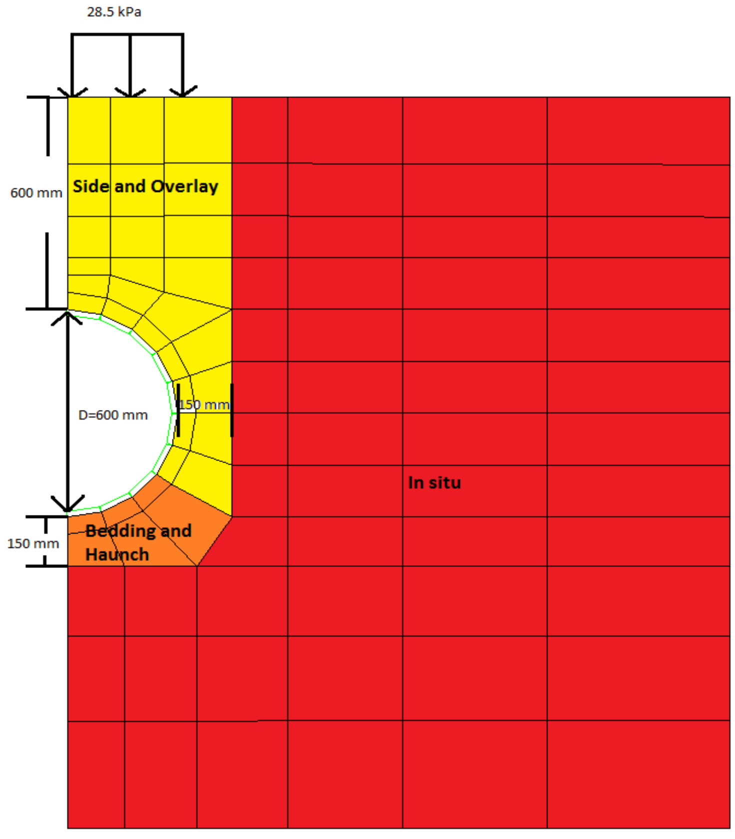


Figure 4.3: Finite Element Mesh for Design Scenarios 600 mm Cover

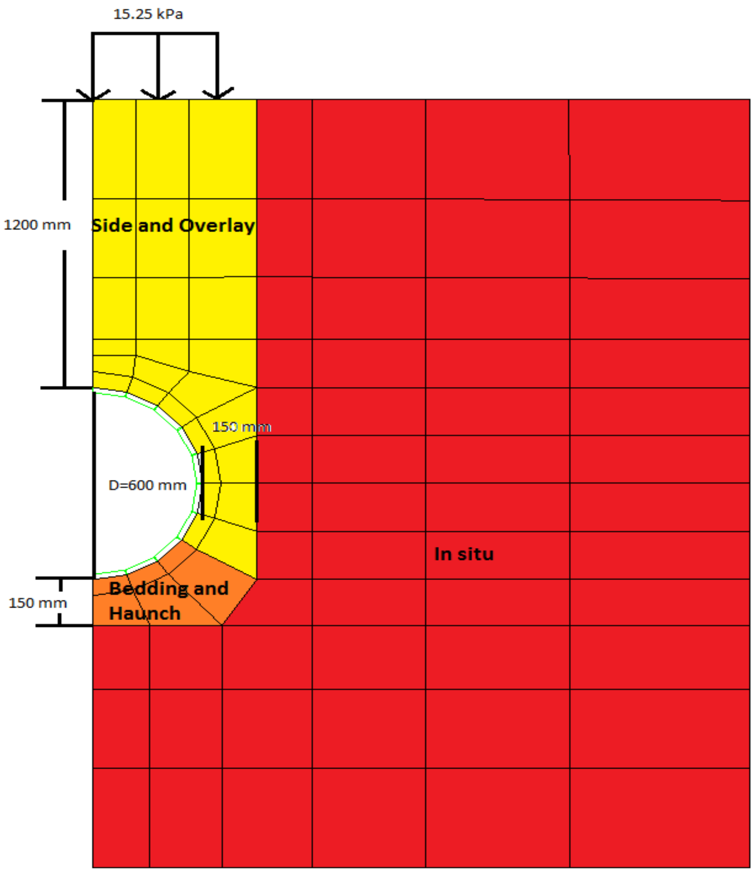


Figure 4.4: Finite Element Mesh for Design Scenarios 1200 mm Cover

Utilising the identified inputs CANDE can then return output data in relation to the stresses, strains, deformations and capacity of the culvert structure and surrounding soil. The output data includes:

- Nodal Coordinates and Displacements (X and Y directions)
- Thrust applied to the beam elements
- Shear applied to beam elements
- Moment applied to beam elements
- Stress applied to soil elements
- Strain applied to soil elements
- Culvert strength parameters and capacity (dependent upon material type)

The output is supplied in a report as well as graphically with the FEM mesh shown with the applied stresses, strains and deflections (Figure 4.5). Plots for the variation of thrust, shear and moment forces are also supplied by CANDE (Figure 4.6). It is important to note that the units of all input and output data are United States imperial and therefore conversion of the data is required for use with SI units.

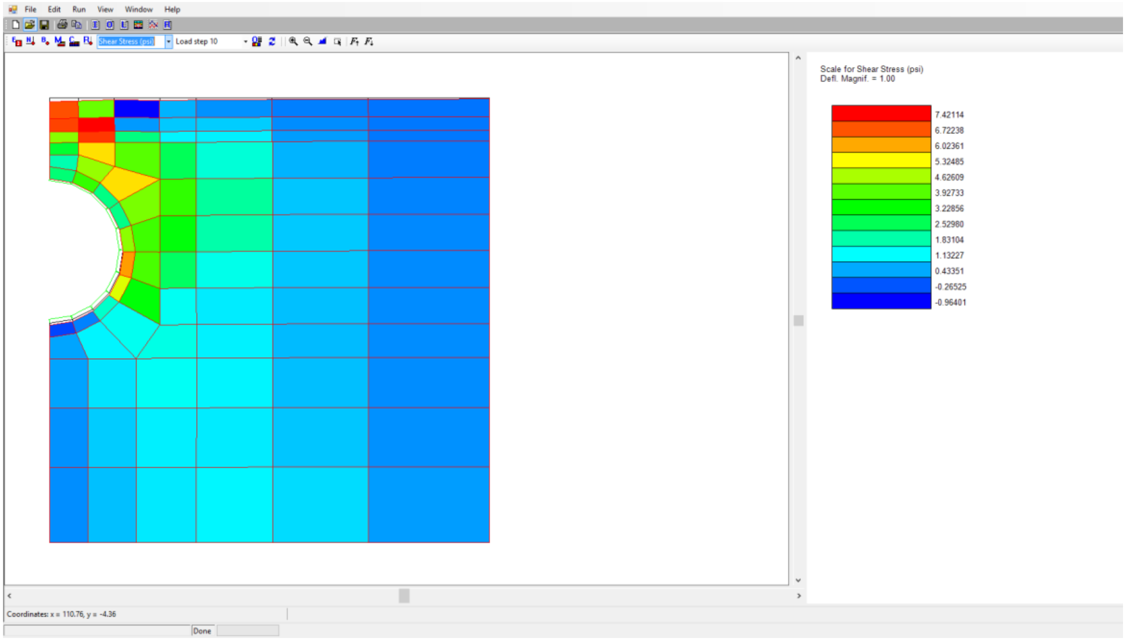


Figure 4.5: Example mesh output

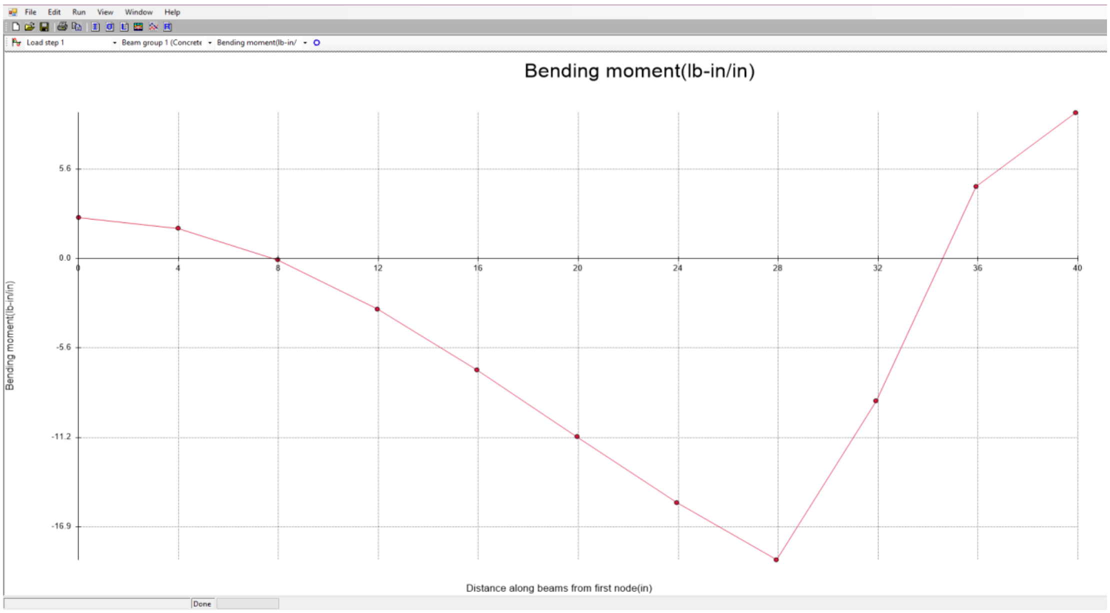


Figure 4.6: Example plot output

4.2 Linear modelling Technique

CANDE utilises a linear elastic soil model in isotropic form in order to analyse the stress distribution through a soil of constant stiffness that is uniform in all directions. This model relies upon the input of the elasticity of the soil medium and the Poisson's ratio to determine the stress and strain distribution throughout the soil. The constitutive matrix (plane strain matric) utilised by CANDE is given as follows (Equation 4.4):

$$\begin{pmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau \end{pmatrix} = \begin{pmatrix} C_{11} & C_{12} & 0 \\ C_{12} & C_{11} & 0 \\ 0 & 0 & C_{33} \end{pmatrix} \begin{pmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma \end{pmatrix}$$

Where;

$$C_{11} = \frac{E(1-\nu)}{(1+\nu)(1-2\nu)}$$

$$C_{12} = \frac{E(\nu)}{(1+\nu)(1-2\nu)}$$

$$C_{33} = \frac{E}{2(1+\nu)}$$

Equation 4.4: Isotropic Linear Elastic Constitutive Matrix (plane strain matrix) (Katona, 2015)

The use of this model is simplified through the use of the GUI which directs the user through the steps/inputs required to analyse the culvert analysis problem. The utilisation of this program for the specified design criteria allow for the investigation into the effects of the soil model and physical parameters upon culvert loading, potential soil movement and backfill soil stress and strain distribution.

The labelling for each design case has been based upon a set of rules. The first two letters signify the analysis method (LE: Linear Elastic, MC: Mohr-Coulomb, D: Duncan), the bedding and backfill type is then given (SM100-CL95 indicates SM100 bedding and CL95 in the side and overlay zone), and finally the trench depth is given in millimetres.

4.3 Duncan Modelling Technique

The Duncan soil model differs from the linear elastic technique by utilising a more realistic stress dependant relationship for Young's and Bulk modulus values. This model uses a hyperbolic function to represent the stress strain relationship (Yoo et al., 2005). The formulation is based upon experimental data obtained from testing soil behaviour by tri-axial testing (Katona, 2015). Duncan found that by equating the deviator stress from these tests (hydrostatic lateral pressure minus axial pressure) to the axial strain the resulting relationship could be modelled by fitting a hyperbolic curve as shown in Equation 4.5 and Figure 4.7.

$$\sigma_1 - \sigma_3 = \frac{\varepsilon}{\frac{1}{E_1} + \frac{\varepsilon}{(\sigma_1 - \sigma_3)_u}}$$

Where;

$$E_1 = Initial\ slope\ of\ Young's\ modulus$$

$$\sigma_1 - \sigma_3 = Deviator\ stress$$

$$(\sigma_1 - \sigma_3)_u = Ultimate\ deviator\ stress$$

$$\varepsilon = Axial\ strain - initial\ hydrostatic\ axial\ strain$$

Equation 4.5: Duncan Hyperbolic Function (Katona, 2015)

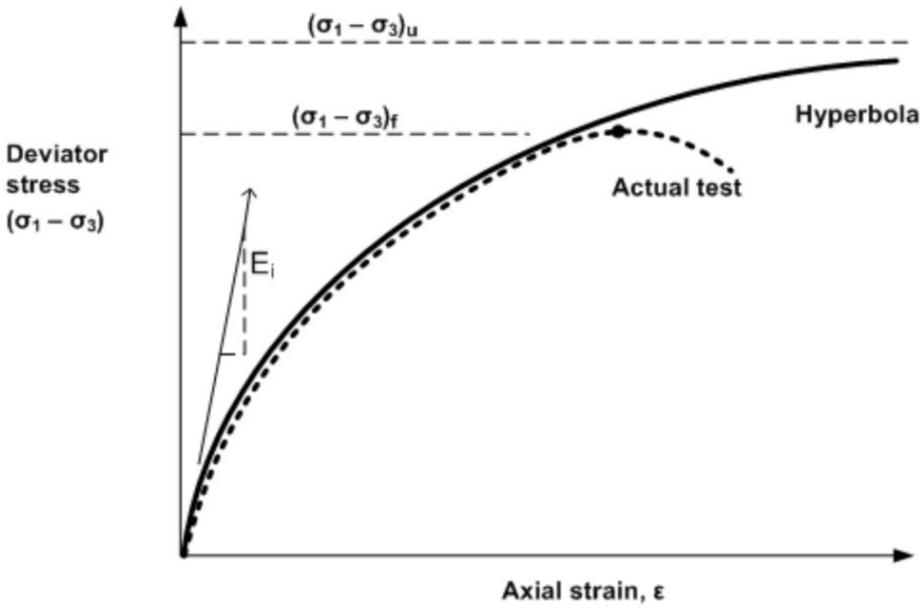


Figure 4.7: Deviator stress vs axial strain (test data vs hyperbolic approximation) (Katona, 2015)

As the hyperbolic curve approaches failure it deviates from actual soil response. Due to this divergence the model utilises an additional parameter which is a ratio of deviator stress at failure to ultimate deviator stress to preserve the curve fit (Katona, 2015). Duncan also introduced a formula for Bulk modulus based upon the experimental data obtained from the tri-axial tests as shown previously in Equation 2.5.

4.4 Mohr Coulomb Modelling Technique

The Mohr Coulomb model, as discussed in the Literature Review, has certain drawbacks for modelling culvert loading due to the high loading and unloading of the structure and the inability of the model to increase soil stiffening with loading. The elastic component of the

Mohr Coulomb model employs similar methods to the elastic model to determine stresses and strains (Equation 4.6).

$$\begin{pmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau \end{pmatrix} = \begin{pmatrix} D_a & D_b & 0 \\ D_b & D_a & 0 \\ 0 & 0 & D_c \end{pmatrix} \begin{pmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma \end{pmatrix}$$

Where;

$$D_a = \frac{E(1 - \nu)}{(1 + \nu)(1 - 2\nu)}$$

$$D_b = \frac{E(\nu)}{(1 + \nu)(1 - 2\nu)}$$

$$D_c = \frac{E}{2(1 + \nu)}$$

$$Constrained\ by\ E > 0\ and\ 0 <= \nu <= 1/2$$

Equation 4.6: Mohr Coulomb Stresses and Strains (Katona, 2015)

Following plastic failure (when τ_{max} is exceeded), however, stresses and strains are computed utilising the flow rule shown in Equation 4.7.

$$\begin{pmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma \end{pmatrix}_p = \Delta\lambda \begin{pmatrix} n_x \\ n_y \\ n_{xy} \end{pmatrix}$$

Where,

$$\begin{pmatrix} n_x \\ n_y \\ n_{xy} \end{pmatrix} = \begin{pmatrix} -(\sigma_x - \sigma_y)/(4Rcos\Phi + \frac{tan\Phi}{2}) \\ (\sigma_x - \sigma_y)/(4Rcos\Phi + \frac{tan\Phi}{2}) \\ \tau_{xy}/Rcos\Phi \end{pmatrix}$$

$$R = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}{}^2}$$

$$\Delta\lambda = The\ magnitud\ of\ plastic\ strain$$

Equation 4.7: Mohr Coulomb Stress Stain Relationship (Katona, 2015)

The Mohr Coulomb method can encounter non convergence when stresses computed by CANDE are statically determinate and exceed the sheer strength of the material. This is of importance for the culvert modelling problem as often live point loads are applied directly to the surface resulting in large shear stresses.

4.5 Validation

Validation of software models is undertaken to ensure that the results obtained are reliable and meet theoretical or tested data. To ensure the program was performing as designed an initial program run was undertaken utilising a tutorial example provided with the program. Checking the results in terms of the accuracy in relation to theoretical or tested outputs was not considered within the scope of the project. This would involve analysis of loads and deformations around culverts in the field or comparisons with alternative analysis software. Both of these scenarios were not feasible given the constraints of time and resources.

The program also provides functionality for the production of the FEM mesh which was utilised within the project. Adjustments to the nodes involved changes to bedding node locations to control bedding thickness and also the addition of surface point loads to represent live loading. The effect of these changes to the mesh was assessed visually to ensure mesh changes did not result in unexpected soil behaviour (in terms of deflections). The live loading was also distributed over several load steps to ensure limited effect upon the results. Convergence of live loading steps was checked to ensure adequacy. The following data presents the analysis of the tutorial example (Table 4.5) and shows there was no deviation.

Table 4.5: Tutorial Validation

	Test (4 load steps)	Tutorial Solution	Percent Difference
Safety Factor for Concrete Crushing	0.776	0.776	0
Safety Factor for Concrete Shear	0.452	0.452	0
Safety Factor for Steel Yielding	0.983	0.983	0

Note: Tutorial data was only given for the results summary.

5 Results

5.1 Linear Elastic Analysis

The model results obtained from CANDE have been presented for the final load step of the 10 steps utilised for modelling with a small deformation analysis mode and standard response data output. The data to be compared are the deflection at the surface directly above the pipe, maximum vertical and horizontal deflections within the backfill and maximum soil stress, thrust, shear and moment. The pipes physical response will also be compared in terms of safety factors against failure in order to assess how the physical response of each backfill type affects the pipe structure. The results for the analysis utilising AS/NZS 3725 have also been presented with safety factors provided.

5.1.1 Deflections

The maximum deflection at the surface and in the Y direction was in the SM100-CL95 (300 mm depth) backfill materials and for the X direction it was SW100-CL95 (300 mm depth). The minimum deflection for the surface and Y direction was the stabilised sand backfill (300 mm depth) and in the X direction the minimum deflection was found in the alternative CLSM mix (300 mm depth). The data generally followed similar trends between each material at different depths with the 300 mm depth having the greatest deflections and the 1200 mm backfill depth having the least. However, the CLSM 28 day material (1200 mm depth) deviated from this trend, exceeding the 600 mm depth for deflection in the X direction. The CLSM materials also had little variation for the maximum deflections in the Y direction. The overall ranges for surface deflection were from 0.6 mm to 18.6 mm. The data is shown below in Figure 5.1, Figure 5.2 and Figure 5.3.

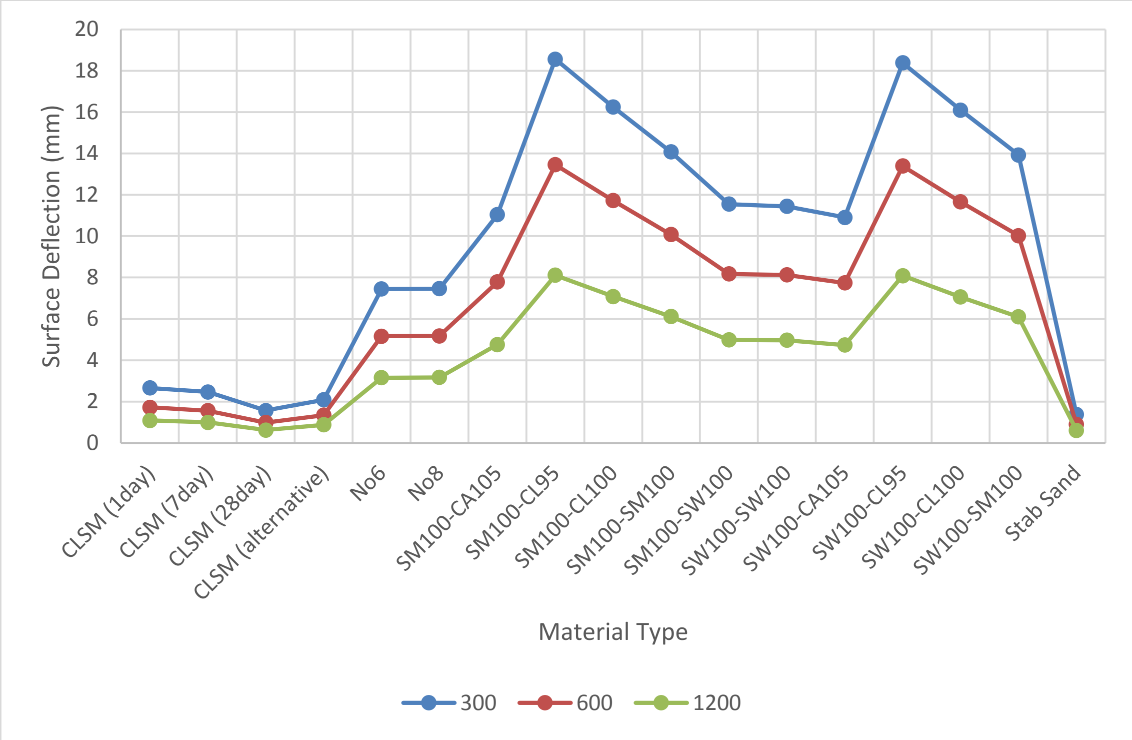


Figure 5.1: Deflection at the Surface

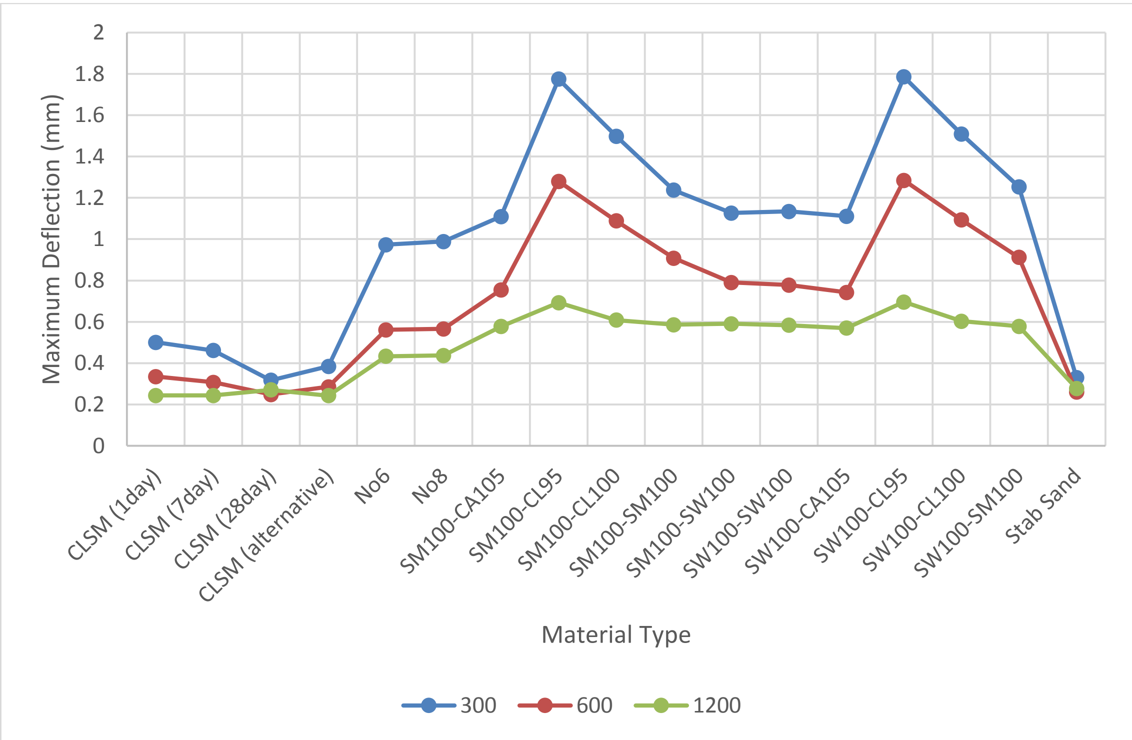


Figure 5.2: Maximum Deflection in the X Direction

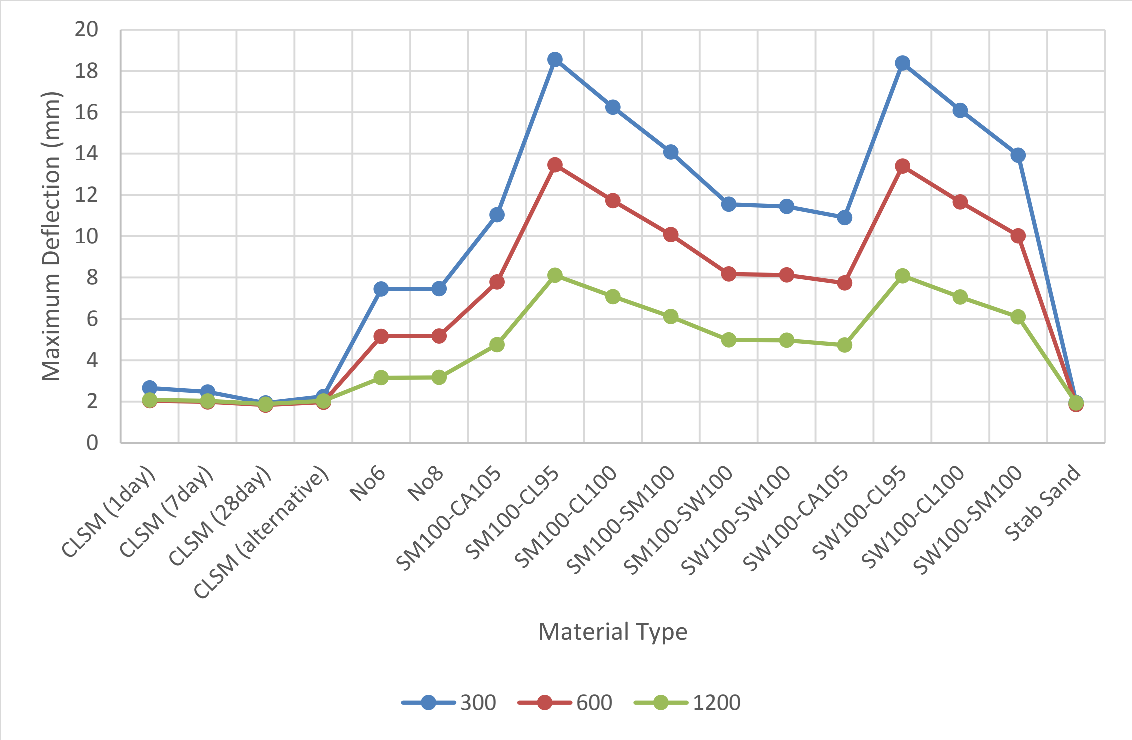


Figure 5.3: Maximum Deflection in the Y direction

5.1.2 Soil Stress, Thrust, Shear and Moment

The material with maximum soil stress and thrust was the stabilised sand (300 mm depth), with values of 346.1 kPa, 43.7 kN/m respectively. Stabilised sand also had the minimum shear of 1.0kN/m and a moment of 0.18 kNm/m at the 1200 mm depth. The minimum thrust was in the SM100-CL95 (1200 mm depth) material at 9.68 kN/m and the maximum shear and moment was SW100-CL95 (300 mm depth) with values 18.8 kN/m and 2.8 kNm/m. The minimum soil stress occurred across several materials at the 300 mm depth with a minimum stress of 80.7 kPa. The data is shown below in Figure 5.4, Figure 5.5, Figure 5.6 and Figure 5.7.

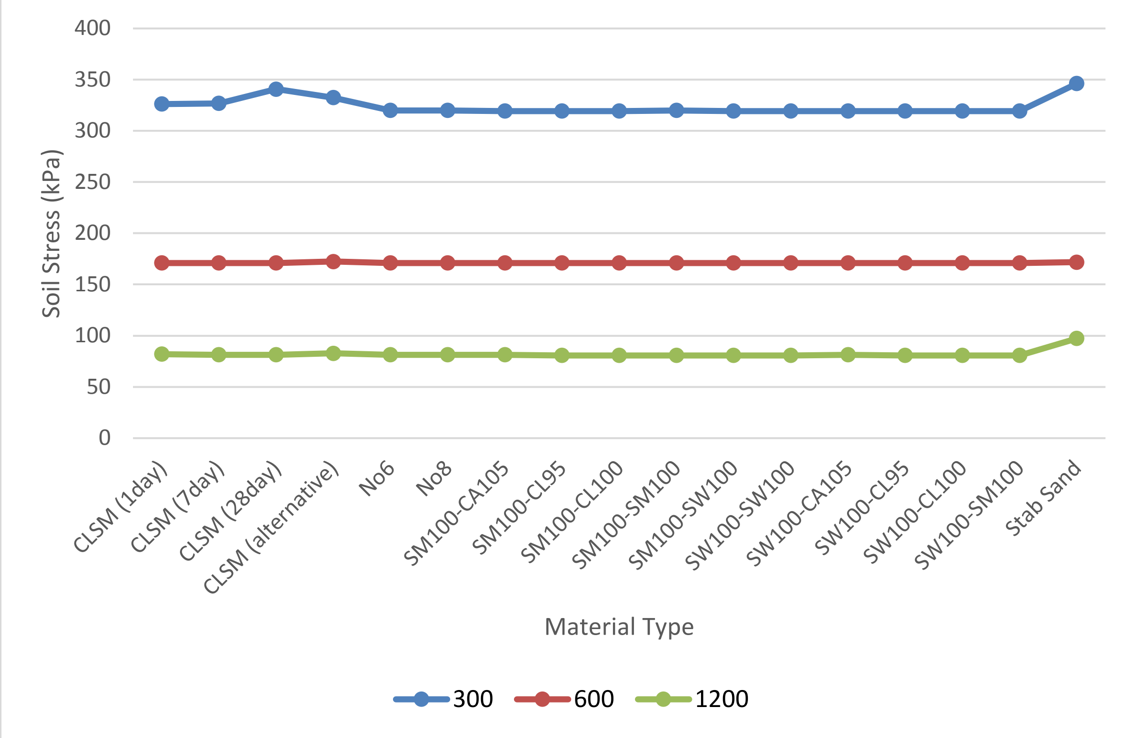


Figure 5.4: Maximum Soil Stress

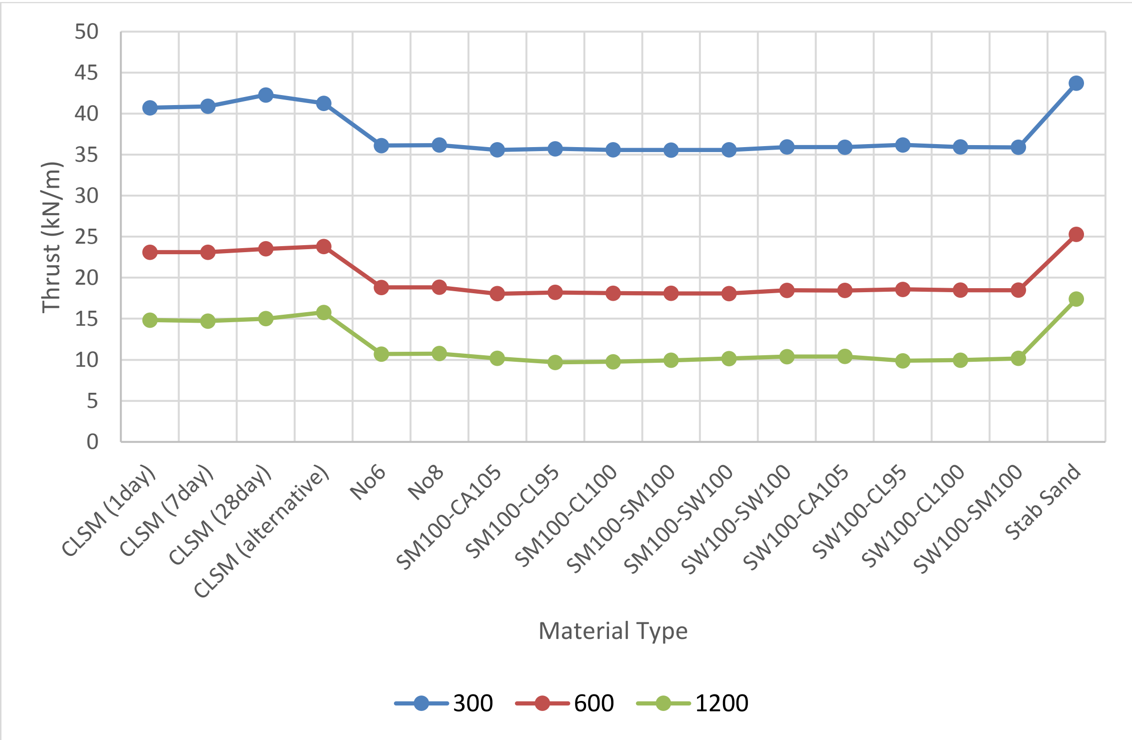


Figure 5.5: Maximum Thrust

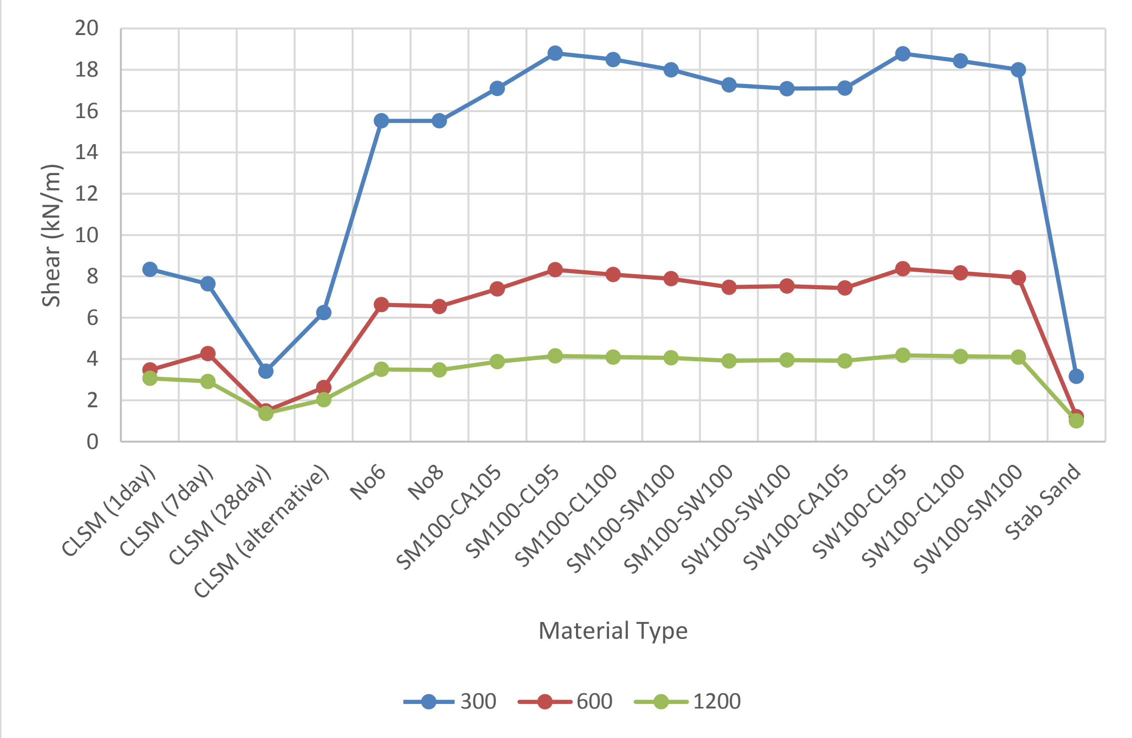


Figure 5.6: Maximum Shear

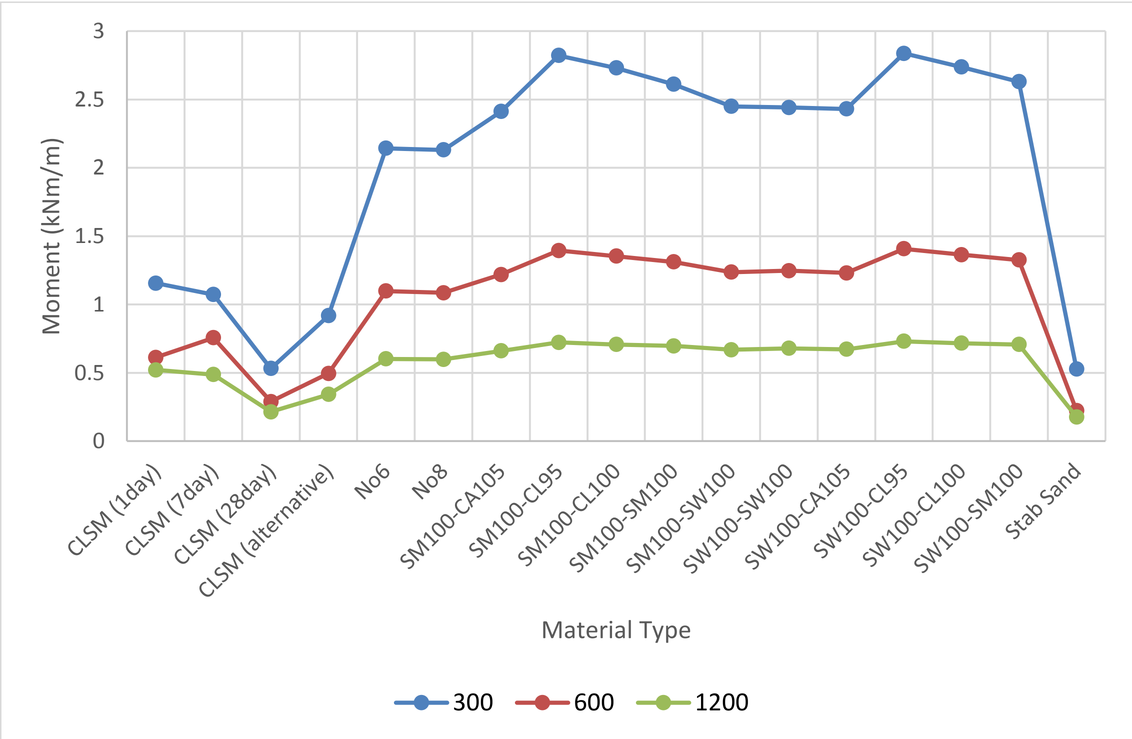


Figure 5.7: Maximum Moment

5.1.3 Safety Factors against Failure

The safety factors against steel yielding within the pipe had a maximum value of 39.7 for the CLSM 28 day backfill at 1200 mm cover and a minimum response of 2.5 for SM100-CL95 (300 mm cover). The safety factor against concrete crushing had similar results with the maximum result being the CLSM 28 day for 1200 mm of cover with a safety factor of 27.2 and the minimum result for stabilised sand at 1200 mm depth of 1.0. The safety factor against shear failure was greatest for the stabilised sand at 23.2 with a 1200 mm cover depth and least for SW100-CL95 at 300 mm cover depth with a safety factor of 1.15. The data for all failure methods followed the trend of being the highest for the 1200 mm cover depth and decreasing to the 300 mm cover depth. However, the stabilised sand at 1200 mm depth had a safety factor of only 1.0 for concrete crushing. The data is shown below in Figure 5.8, Figure 5.9 and Figure 5.10.

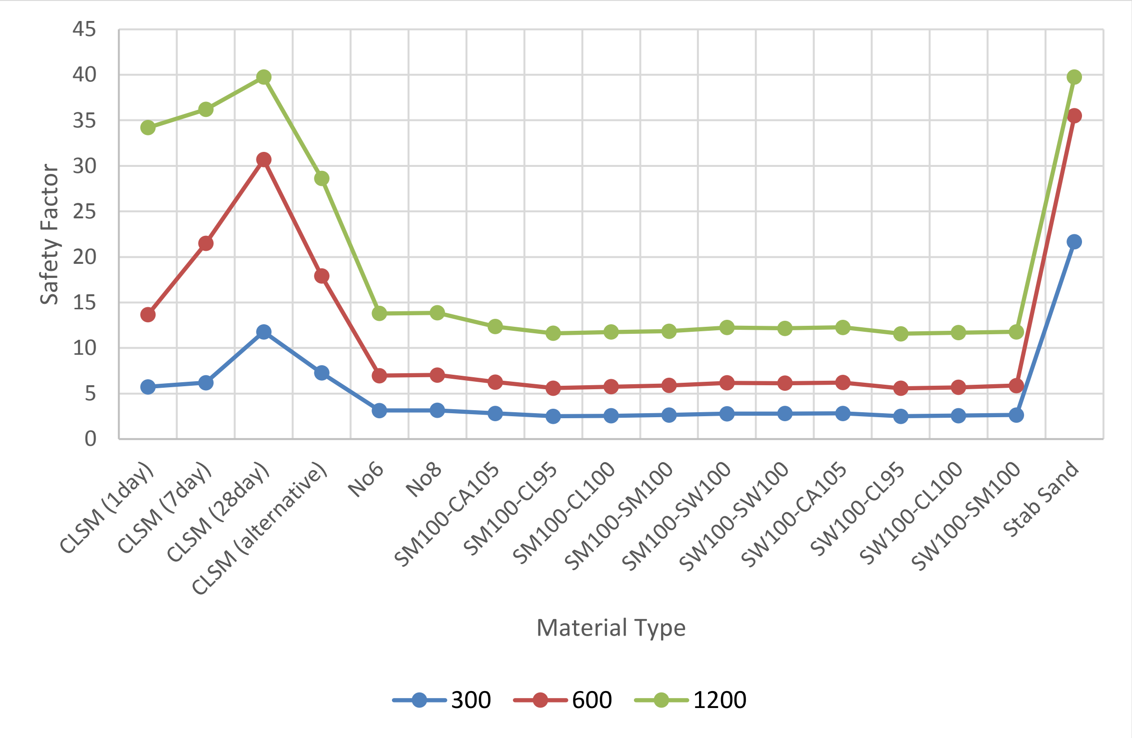


Figure 5.8: Safety Factor Steel Yielding

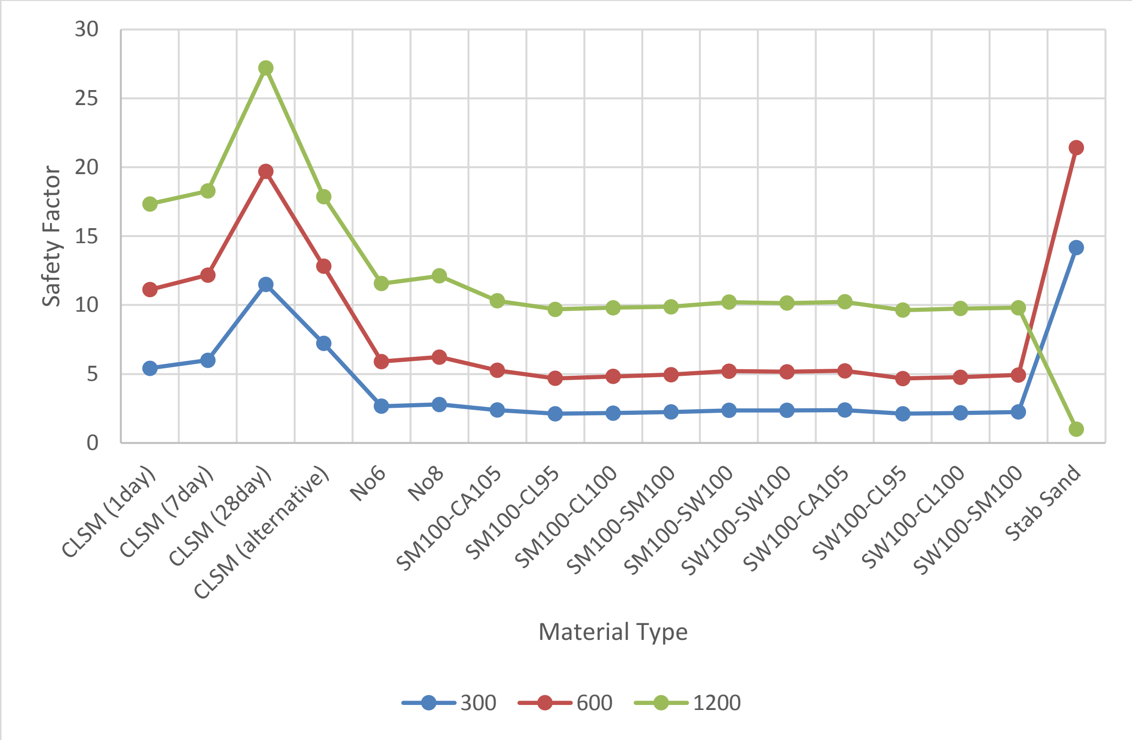


Figure 5.9: Safety Factor Concrete Crushing

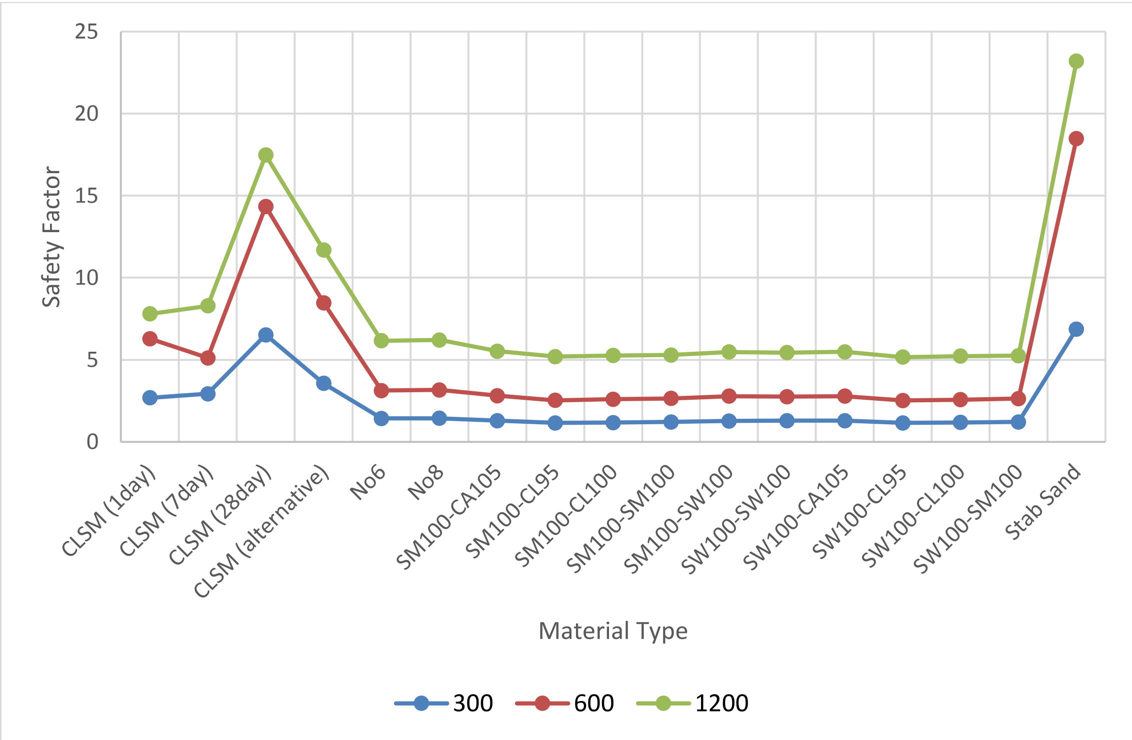


Figure 5.10: Safety Factor Shear Failure

5.2 Analysis of Reduced Width CLSM

One of the benefits of using CLSM is its ability to be self-compacting which can enable reduced trench widths. A comparison of the ability to have reduced trench widths has been made for the linear elastic analysis method with the distance from the edge of the trench to the pipe wall being 100 mm rather than 150 mm.

5.2.1 Deflections

The model results for the reduced trench width of 100 mm, when utilising CLSM, is outlined below. Results for the CLSM 150 mm trench widths have been included in the following figures (Figure 5.11, Figure 5.12 and Figure 5.13) for comparison, with the legend displaying the trench depth/trench width. The deflections for the 300 mm cover depth were the largest, however, there were no identifiable trends between the 150 and 100 mm trench widths. For the 600 mm cover depth the 150 mm trench width always had greater deflections than the 100 mm width. The 1200 mm cover depth, like the 300 mm, had no identifiable trends between differing trench widths. For the deflection in the X direction the 100 mm trench width generally had lower deflections, however, this was not consistent throughout each of the materials.

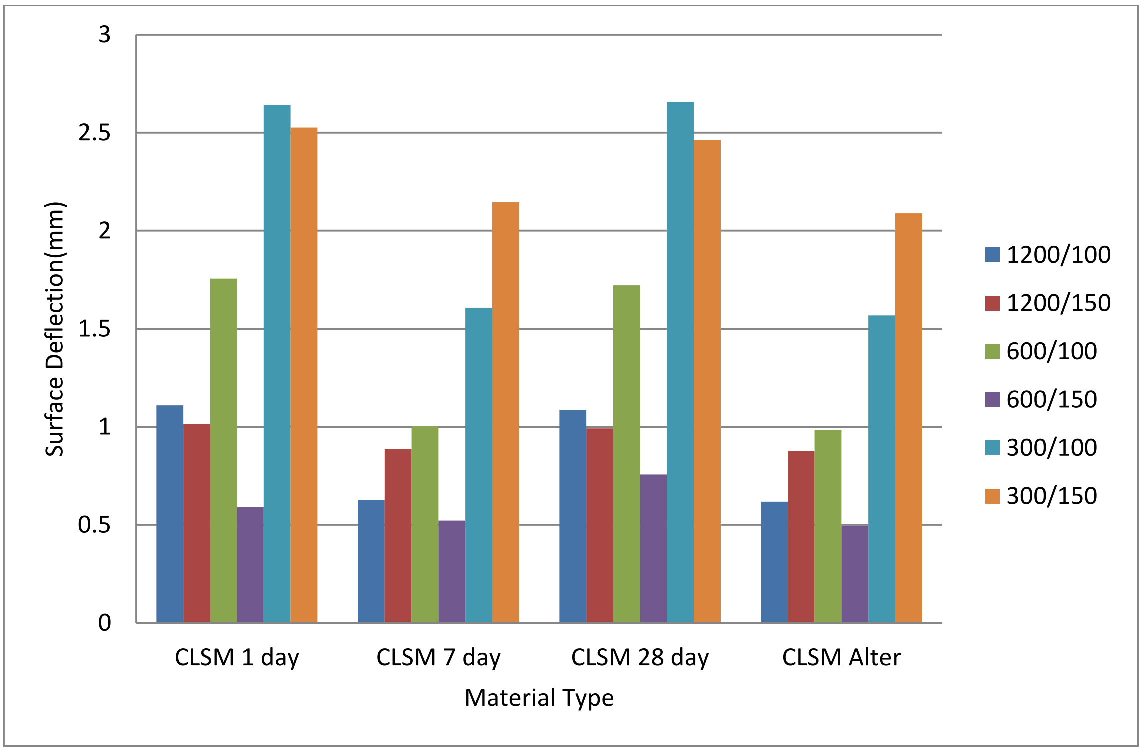


Figure 5.11: Deflection at the Surface

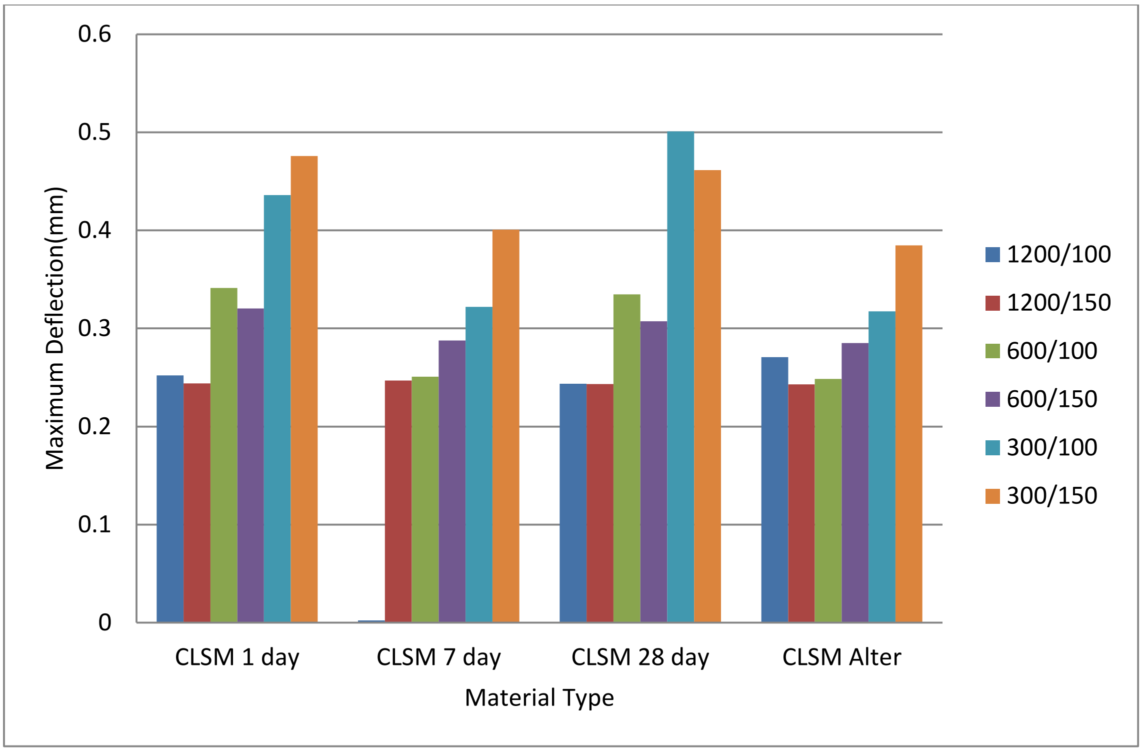


Figure 5.12: Maximum Deflection in the X Direction

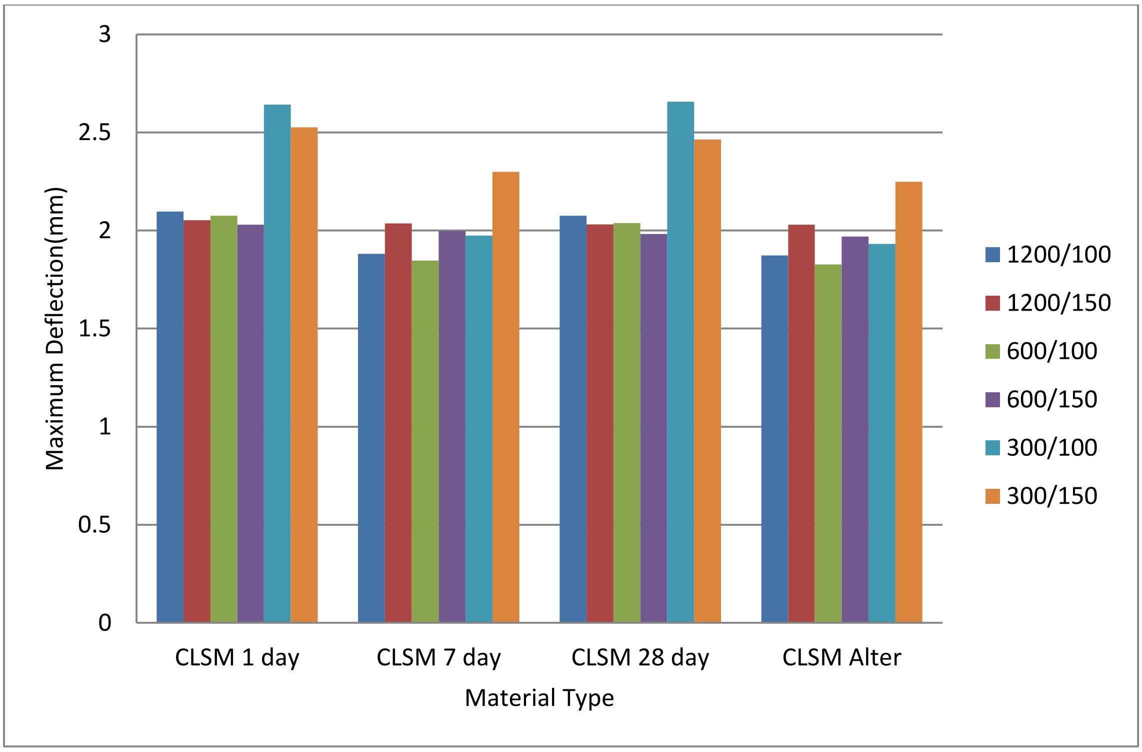


Figure 5.13: Maximum Deflection in the Y Direction

5.2.2 Soil Stress, Thrust, Shear and Moment

When analysing the shear stress and the thrust no major differences were identified between the changes in trench width. For maximum shear and moment there were no visible trends that were consistent across material types (Figure 5.14, Figure 5.15, Figure 5.16 and Figure

5.17). The CLSM alternative material resulted in increased soil stress and thrust and reduced shear and moment for the 100mm trench widths. The other materials were not as consistent across the various cover depths with the comparison being dependent upon cover depth.

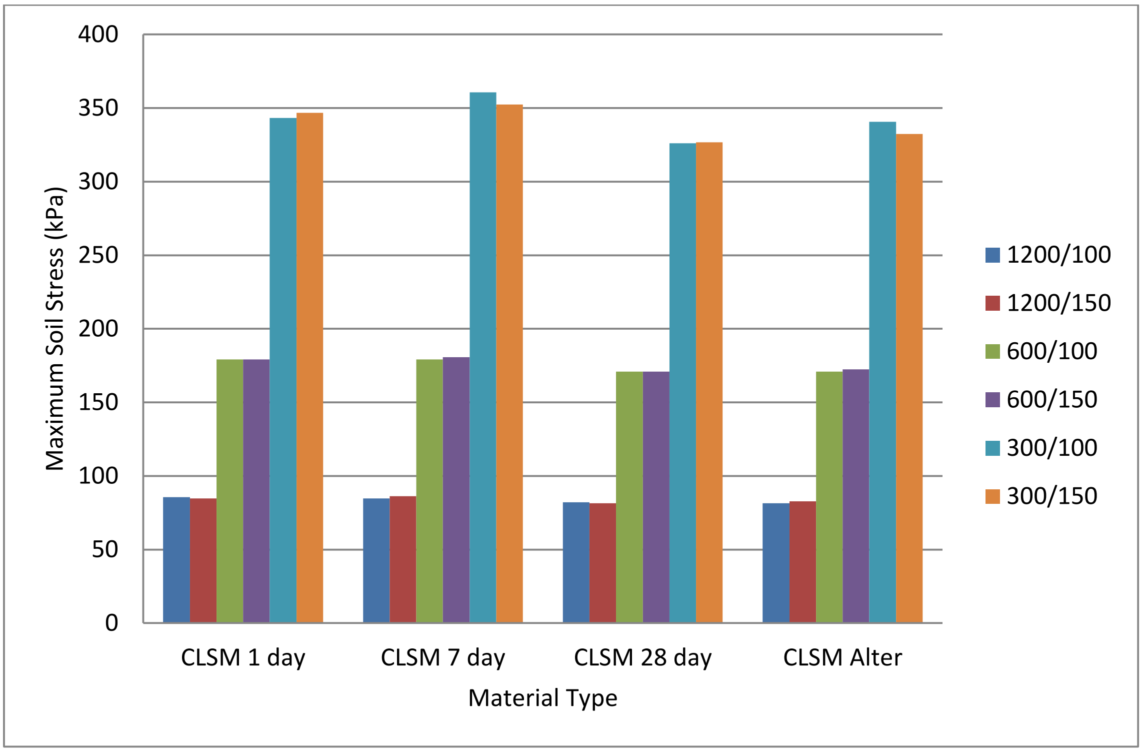


Figure 5.14: Maximum Soil Stress

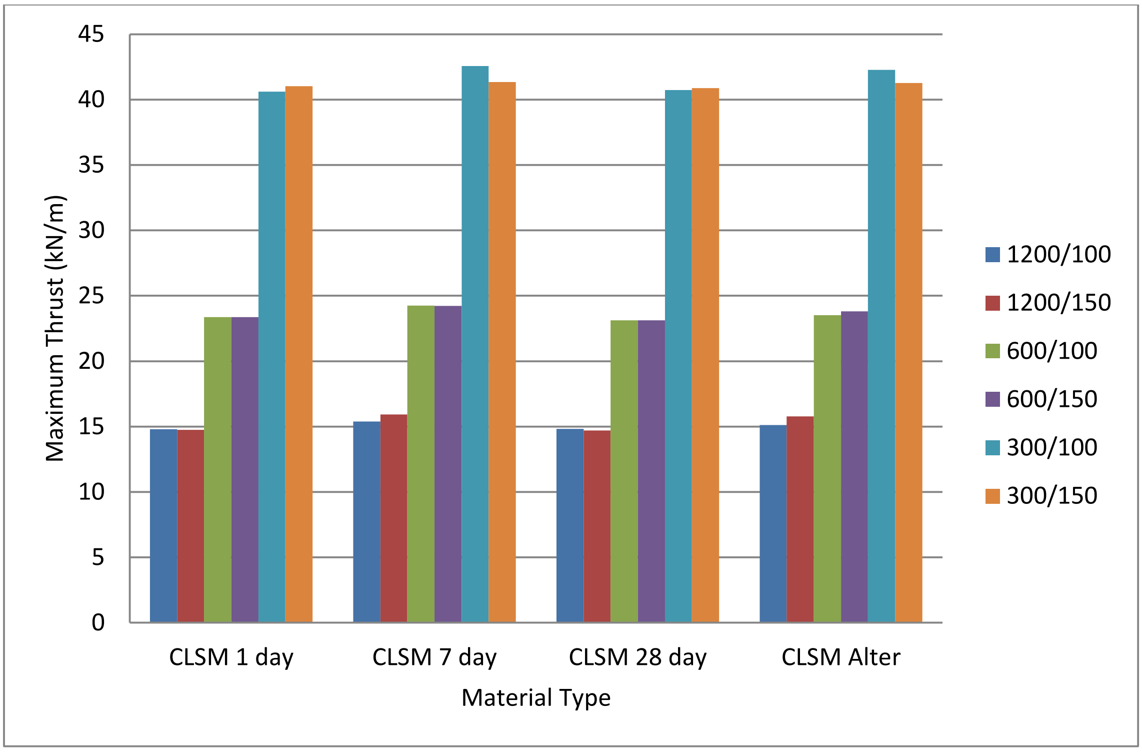


Figure 5.15: Maximum Thrust

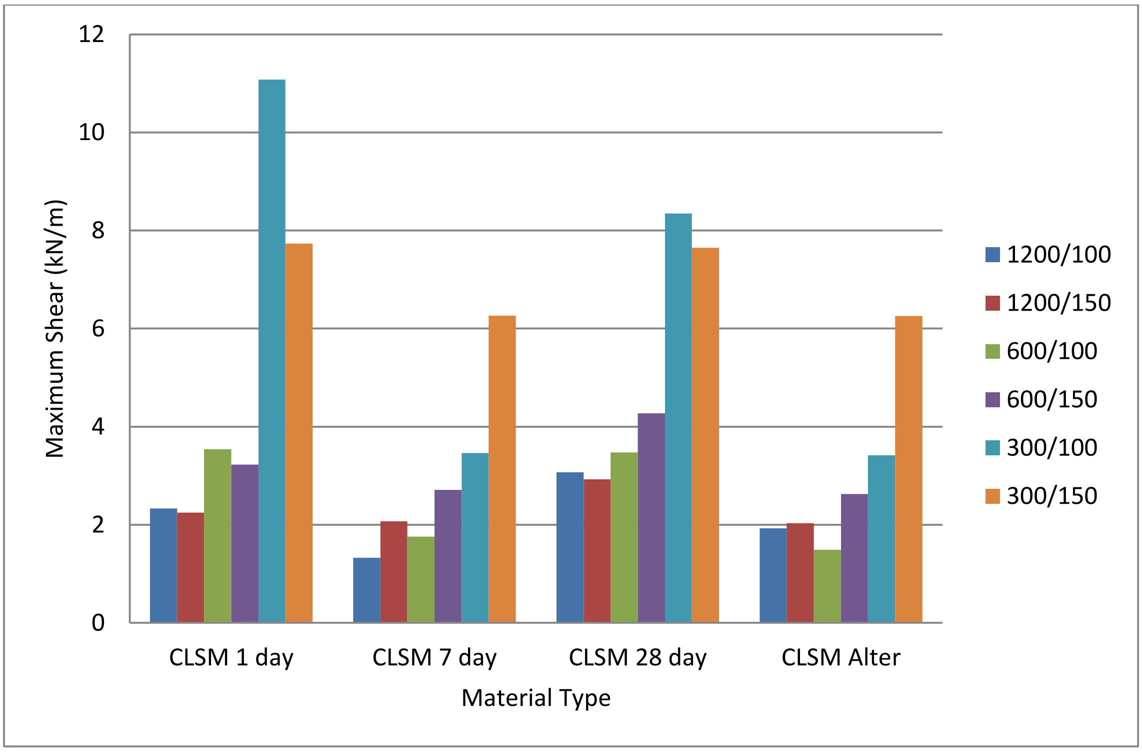


Figure 5.16: Maximum Shear

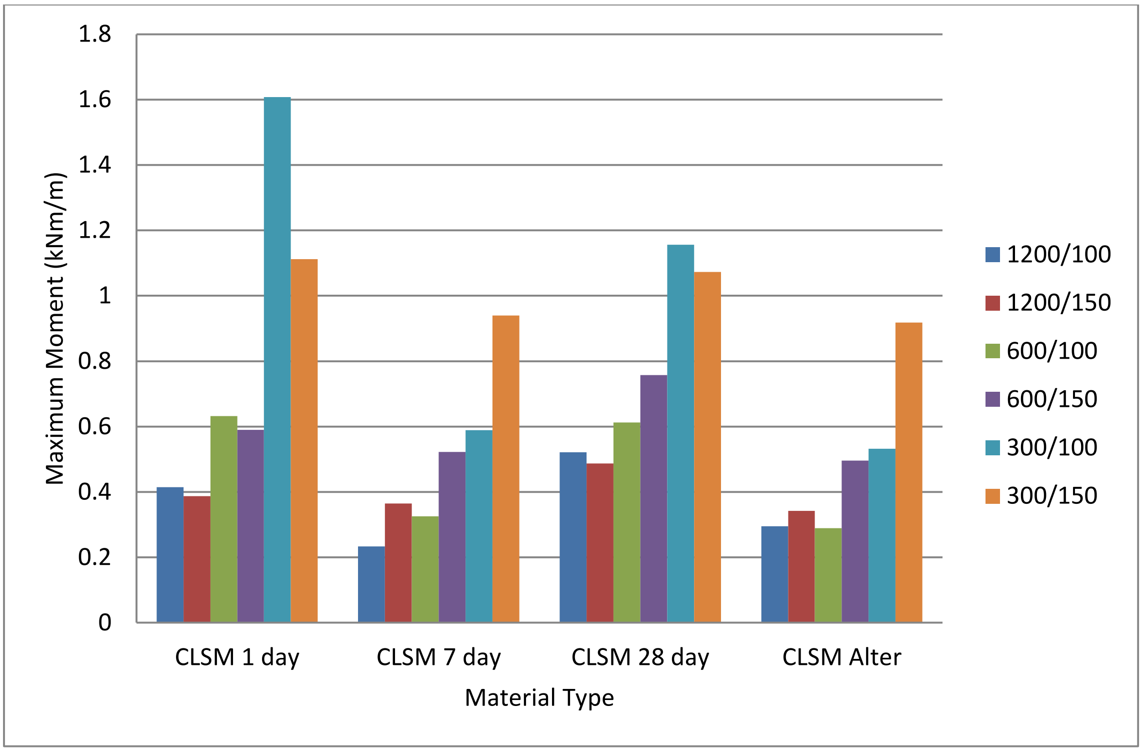


Figure 5.17: Maximum Moment

5.2.3 Safety Factors against Failure

For the safety factors against failure once again there were no consistent trends between trench widths across all material properties (Figure 5.18, Figure 5.19 and Figure 5.20). However, the CLSM alternative material resulted in higher safety factors for the 100 mm trench width with the opposite being true for all other CLSM materials (except for shear failure which had varying responses depending upon cover depth).

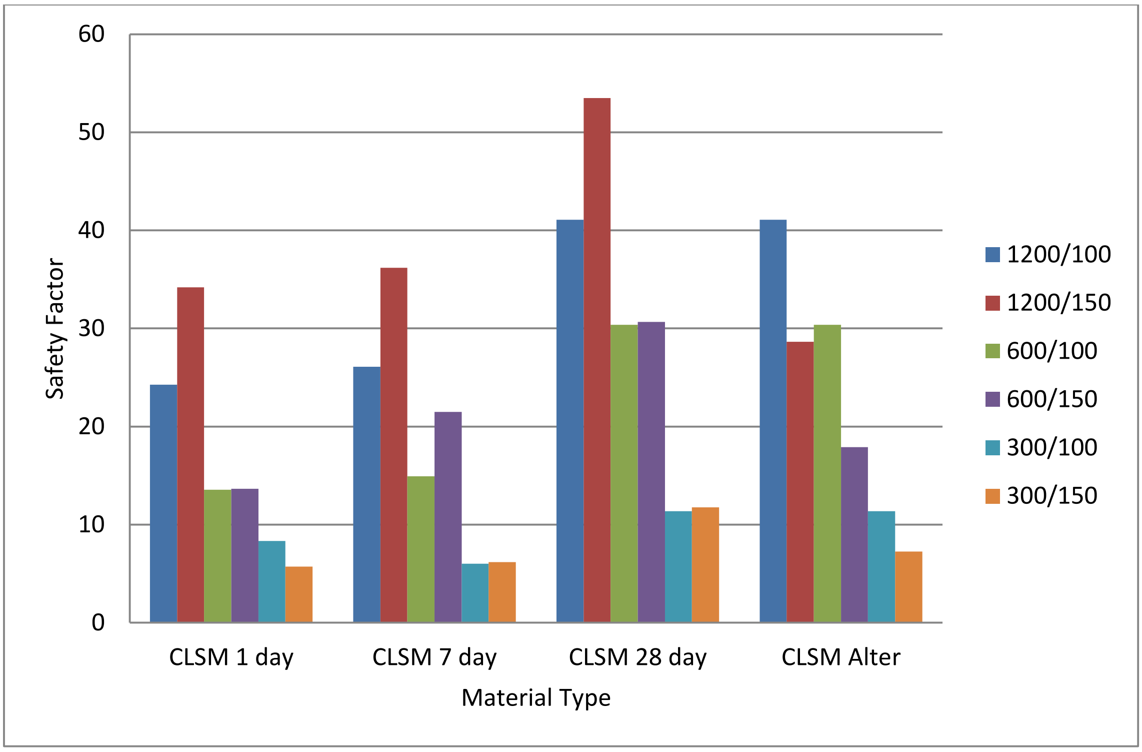


Figure 5.18: Safety Factor Steel Yielding

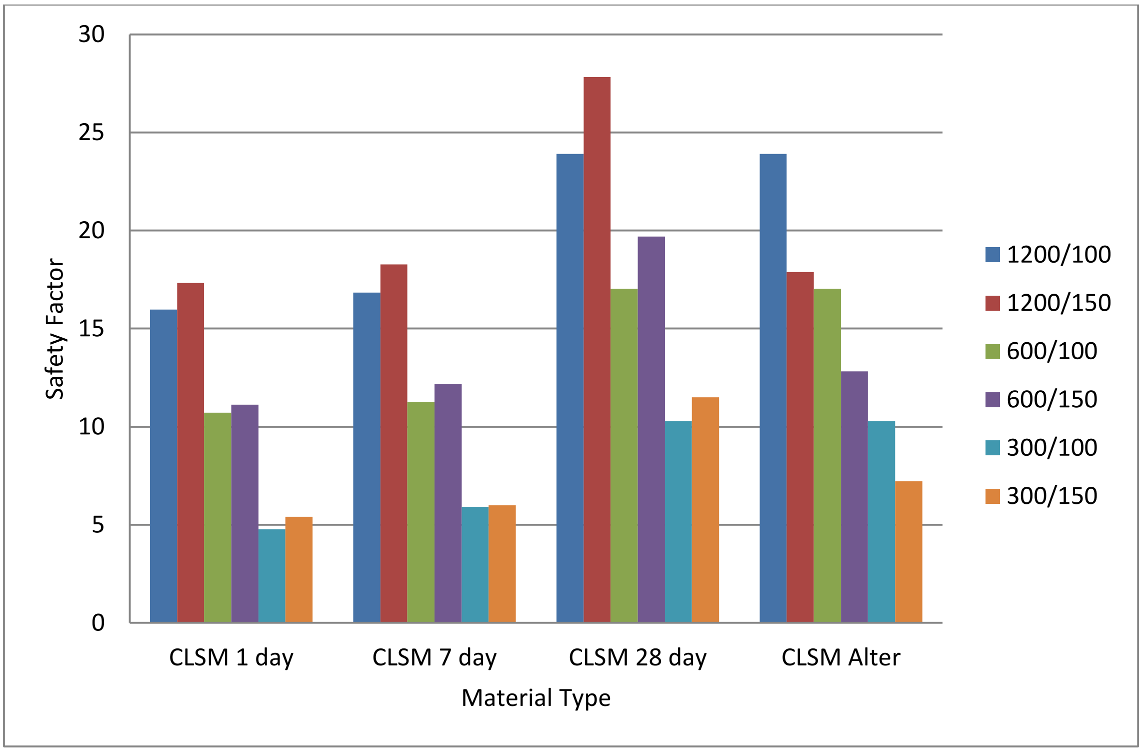


Figure 5.19: Safety Factor Concrete Crushing

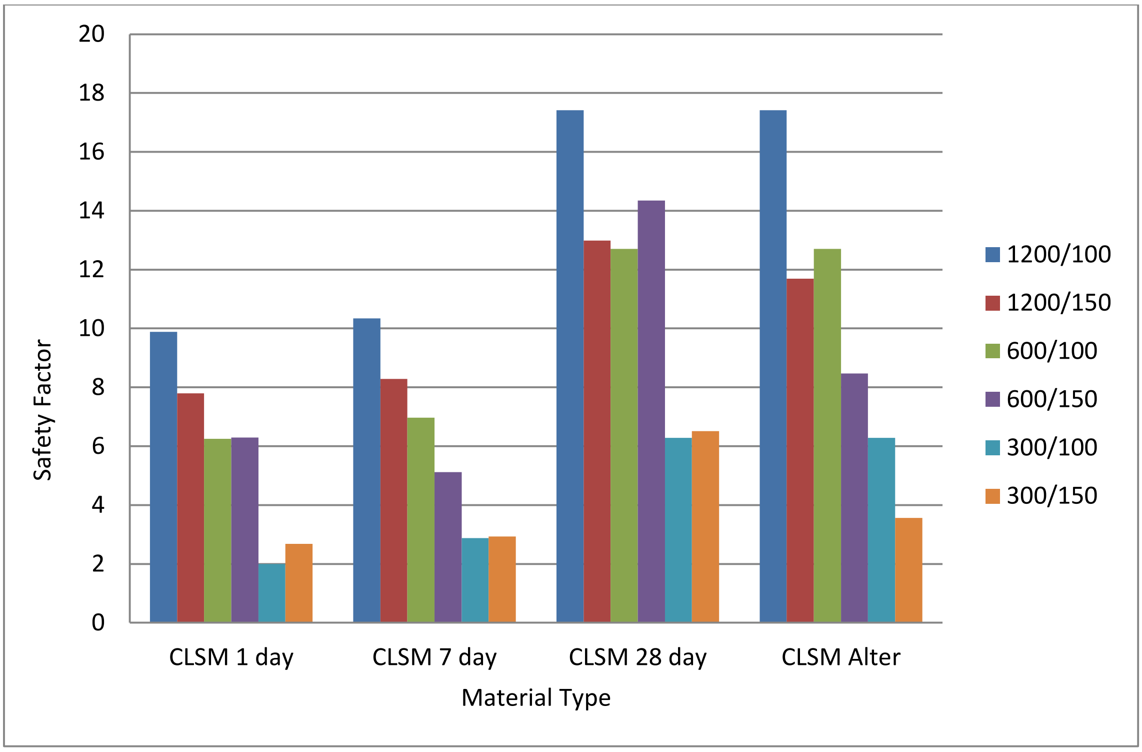


Figure 5.20: Safety Factor Shear Failure

5.3 Duncan Model

The model results obtained from CANDE have been presented for the final load step of the 10 steps utilised for modelling with a small deformation analysis mode and standard response data output. The data to be compared are the deflection at the surface directly above the pipe, maximum vertical and horizontal deflections within the backfill and maximum soil stress, thrust, shear and moment. The pipes physical response will also be compared in terms of safety factors against failure in order to assess how the physical response of each backfill type affects the pipe structure. The Duncan model analysis only utilised the granular materials as parameters for the alternative backfills are not available.

5.3.1 Deflections

The maximum surface deflection (Figure 5.21) matched the maximum Y deflections (Figure 5.23) with the largest deflections encountered in the SM100-CL95 and SW100-CL95 bedding and backfill. The maximum deflections were approximately 48 mm. The lowest deflections occurred throughout the remaining granular materials with SM100-SM100 and SW100-SM100 returning the lowest values of approximately 3.1 mm. The trends across depth indicate that with lower cover there are higher deflections.

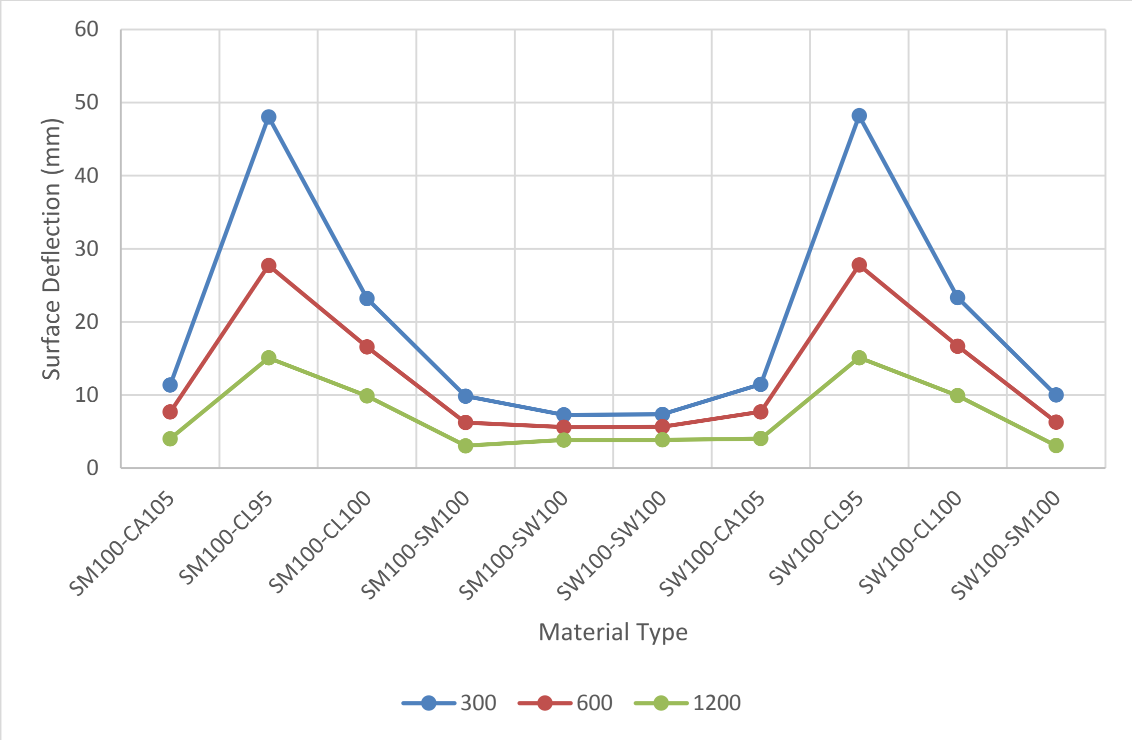


Figure 5.21: Deflection at the Surface

The X deflections (Figure 5.22) followed similar trends to the Y deflection (Figure 5.23), with the highest deflections from the CL95 material of approximately 8.4 mm. The lowest deflections are again obtained by the SM100-SM100 and SW100-SM100 with values of approximately 0.4 mm.

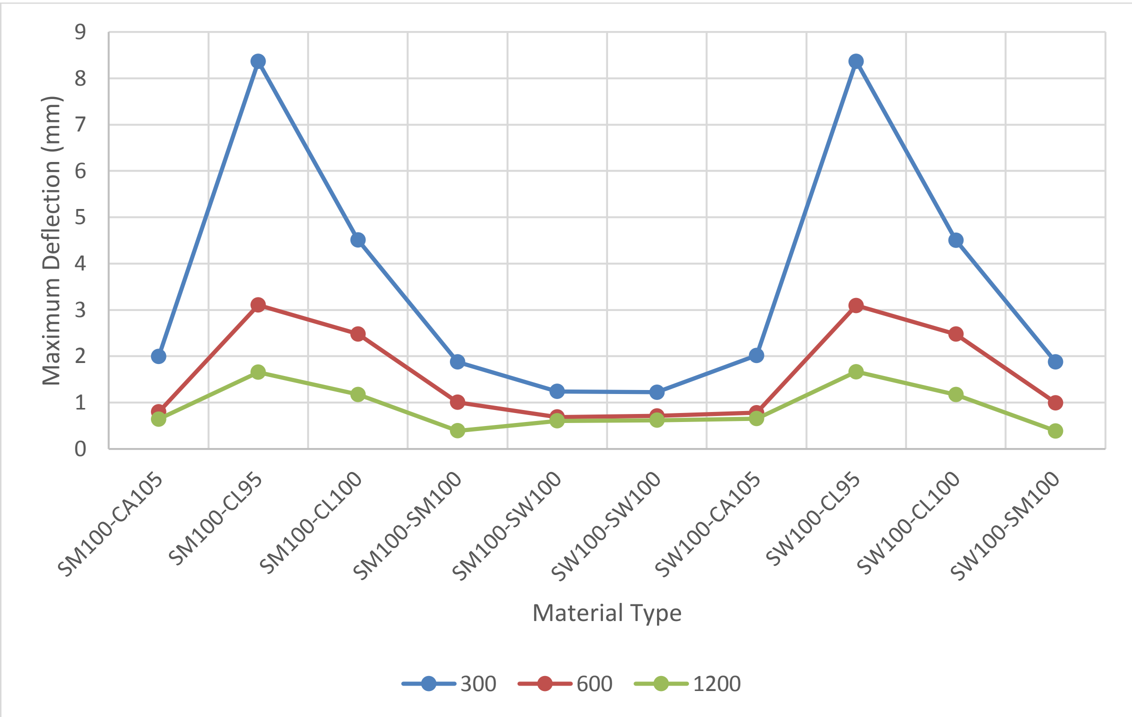


Figure 5.22: Maximum Deflection in the X Direction

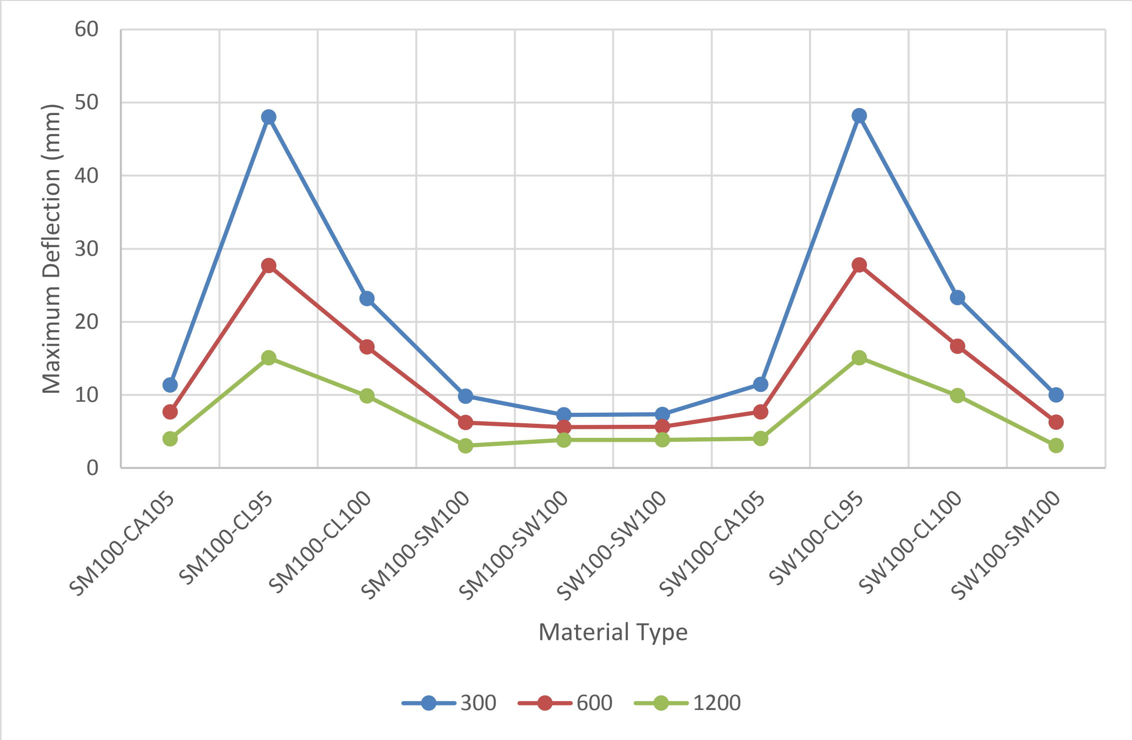


Figure 5.23: Maximum Deflection in the Y Direction

5.3.2 Soil Stress, Thrust, Shear and Moment

The maximum soil stress, thrust, shear and moment all follow the trend of having lower values for higher cover (e.g 1200 mm cover). The results give a maximum stress (Figure 5.24), from the SM100-SM100 (300 mm cover), of 340 kPa and a minimum stress of 80.0 kPa, from the 1200 mm cover SM100-CL95 and SW100-CL95. The maximum thrust occurred in the SM100-CL100 (300 mm cover) and was 38.4 kN/m, with the minimum thrust from the SW100-CL95 (1200 mm cover) of 9.4 kN/m (Figure 5.25). From Figure 5.26 it is shown that the maximum shear was from the SM100-CL95 (300 mm cover) with 19.4 kN/m and a minimum shear of 3.4 kN/m was from the SW100-CA105 (1200 mm cover). SM100-CL95 also had the highest moment of 3.1 kNm/m at 300 mm cover and the minimum moment of 0.58 kNm/m occurred in the SW100-CA105 at 1200 mm cover (Figure 5.27).

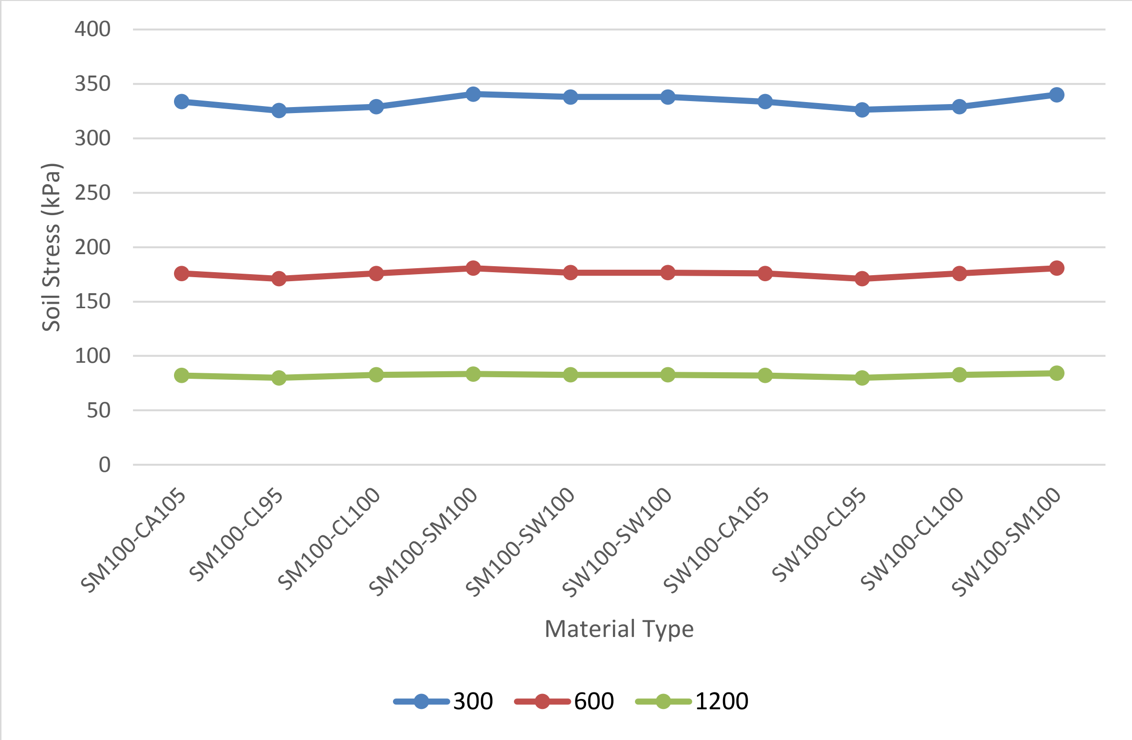


Figure 5.24: Maximum Soil Stress

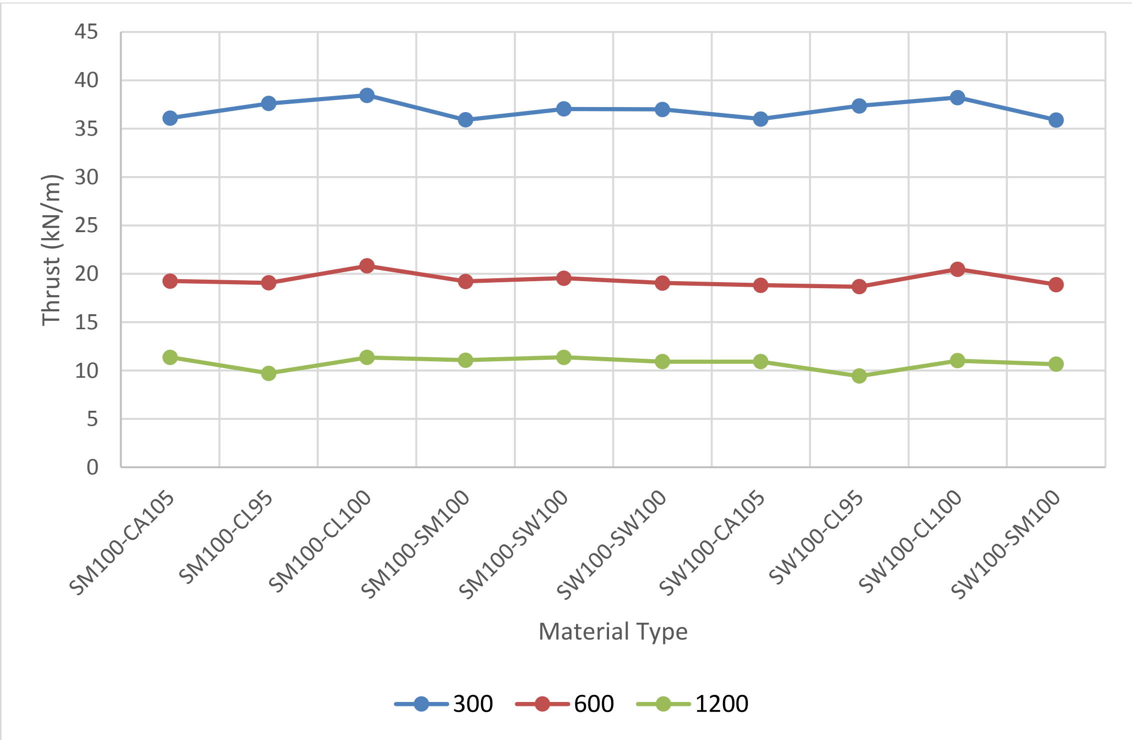


Figure 5.25: Maximum Thrust

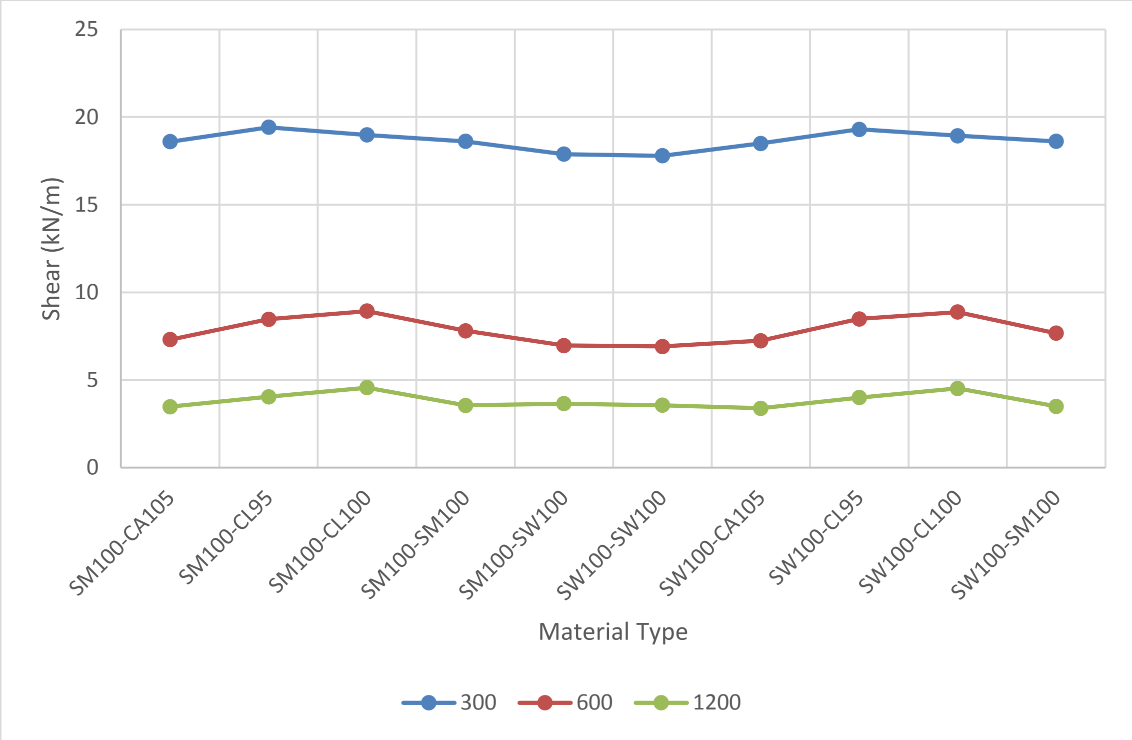


Figure 5.26: Maximum Shear

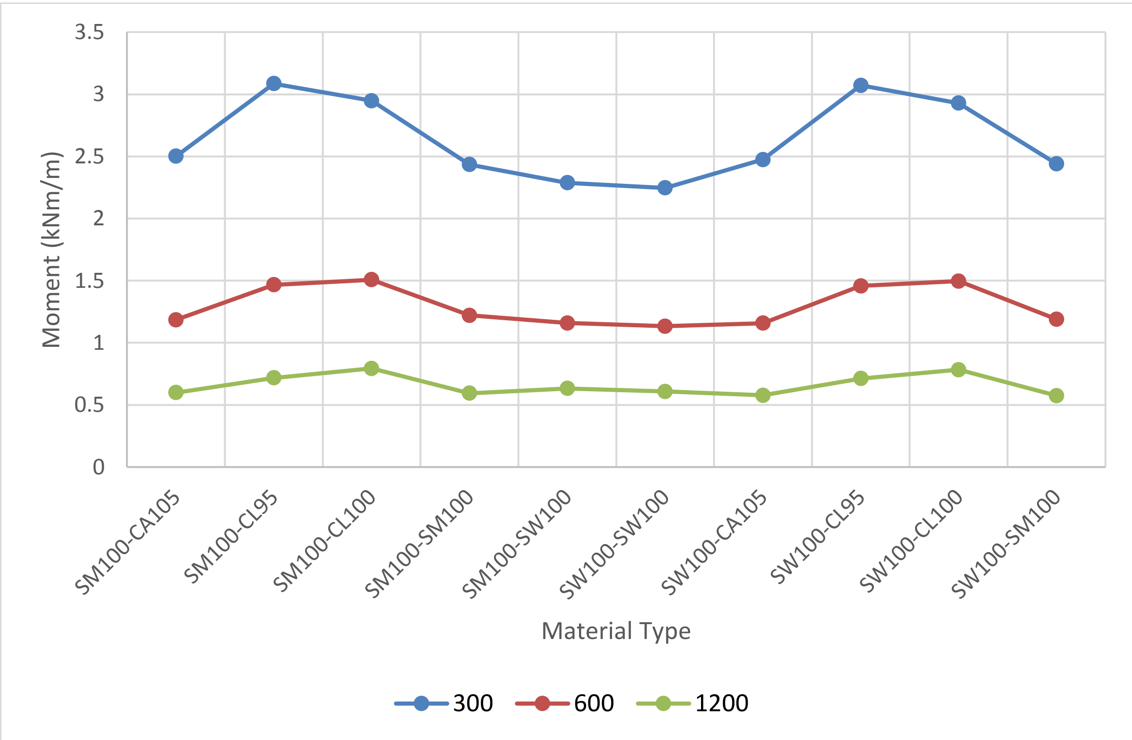


Figure 5.27: Maximum Moment

5.3.3 Safety Factors against Failure

From the Duncan Model the safety factors against steel yielding indicated that there was no stress in the reinforcing, hence this has not been included in the following figures. Failure from concrete crushing was less likely than from shear. However, from Figure 5.28 we can see

SW100-SM100 had the highest safety factor of 10.8 and the minimum occurred from SM100-CL95 with a safety factor of 1.89. The greatest safety factor against shear failure was 6.25 from SW100-CA105 and the minimum was 1.09 from SM100-CL95 (Figure 5.29). Again these values indicate higher safety factors for the 1200 mm cover depths and lower values for the 300 mm cover depths with the same trends across different cover depths.

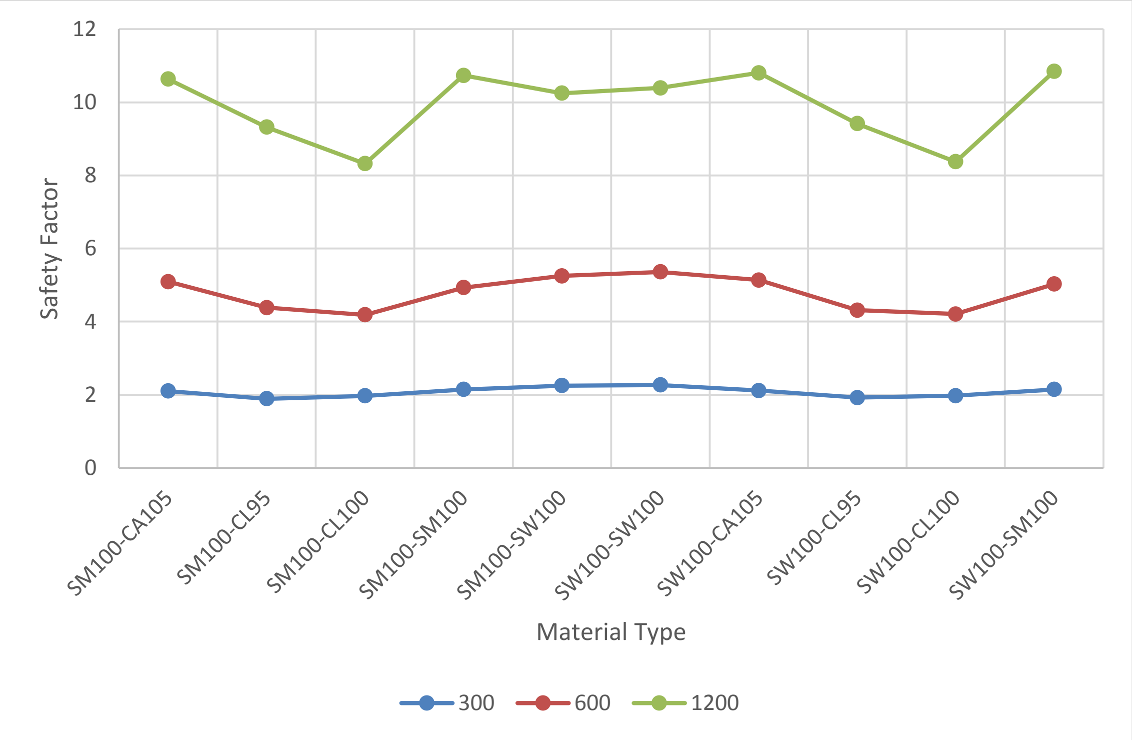


Figure 5.28: Safety Factor Concrete Crushing

Figure 5.29: Safety Factor Shear Failure

5.4 Mohr Coulomb Model

The model results obtained from CANDE have been presented for the final load step of the 10 steps utilised for modelling with a small deformation analysis mode and standard response data output. The data to be compared are the deflection at the surface directly above the pipe, maximum vertical and horizontal deflections within the backfill and maximum soil stress, thrust, shear and moment. The pipes physical response was also compared in terms of safety factors against failure in order to assess how the physical response of each backfill type affects the pipe structure. The Mohr Coulomb analysis did not utilise the three examples of CLSM (1 day, 7 day and 28 day strengths) as cohesion and the angle of internal friction parameters were not available.

5.4.1 Deflections

The values for surface deflection (Figure 5.30) and deflection in the Y direction followed similar trends but there were a few inconsistencies (for example SW100-CA105 surface deflection equalled 19.3 mm whereas the maximum Y deflection was 22.2 mm). Again similar trends occurred with maximum deflections occurring for lower cover depths. From Figure 5.30 the largest deflection at the surface occurred from the 600 mm cover depth stabilised sand at 24.2 mm and the least deflection was from the CLSM with 0.8 mm (excluding the 300 mm SW100-SW100 which had a negative surface deflection and unusual maximum X and Y deflections and was therefore removed).

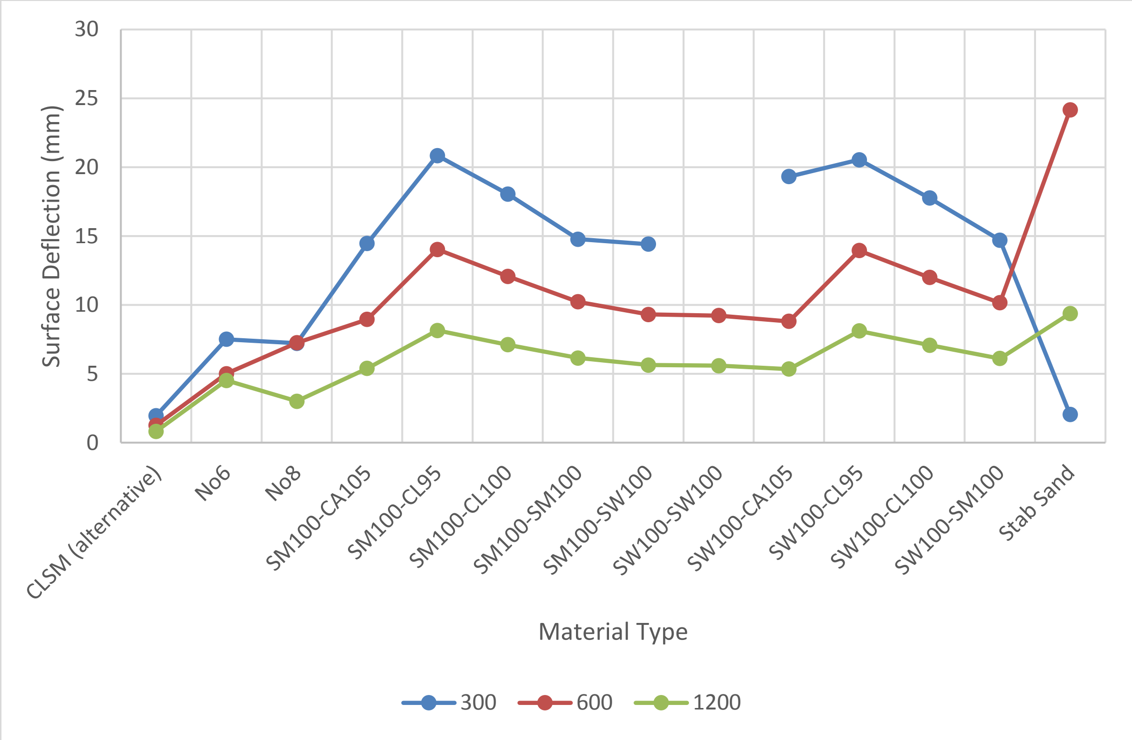


Figure 5.30: Deflection at the Surface

The deflections in the X direction (Figure 5.31) indicated a maximum value for the stabilised sand with a cover of 600 mm and a value of 21.9 mm. The minimum value obtained was from the 1200 mm cover CLSM with a value of 0.23 mm. There was little deviation between the surface deflections and the maximum deflections in the Y direction (Figure 5.32). The SM100-SW100 and the stabilised sand were exceptions to the general trend of increasing deflection with reducing cover.

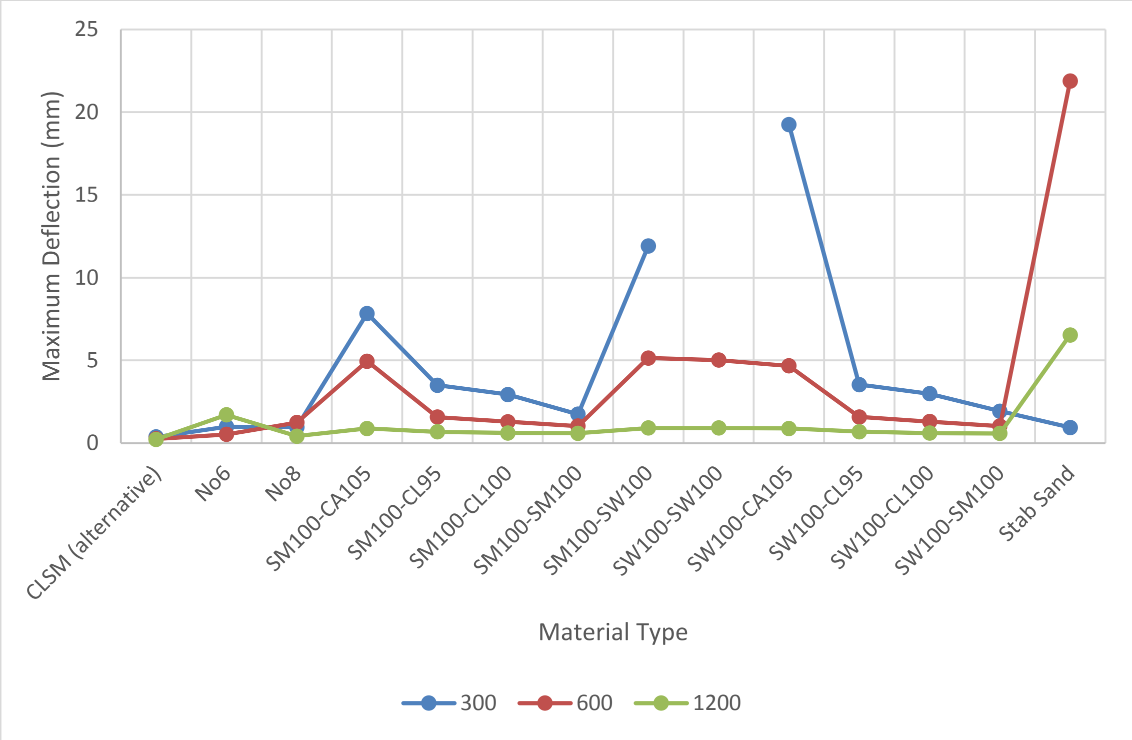


Figure 5.31: Maximum Deflection in the X Direction

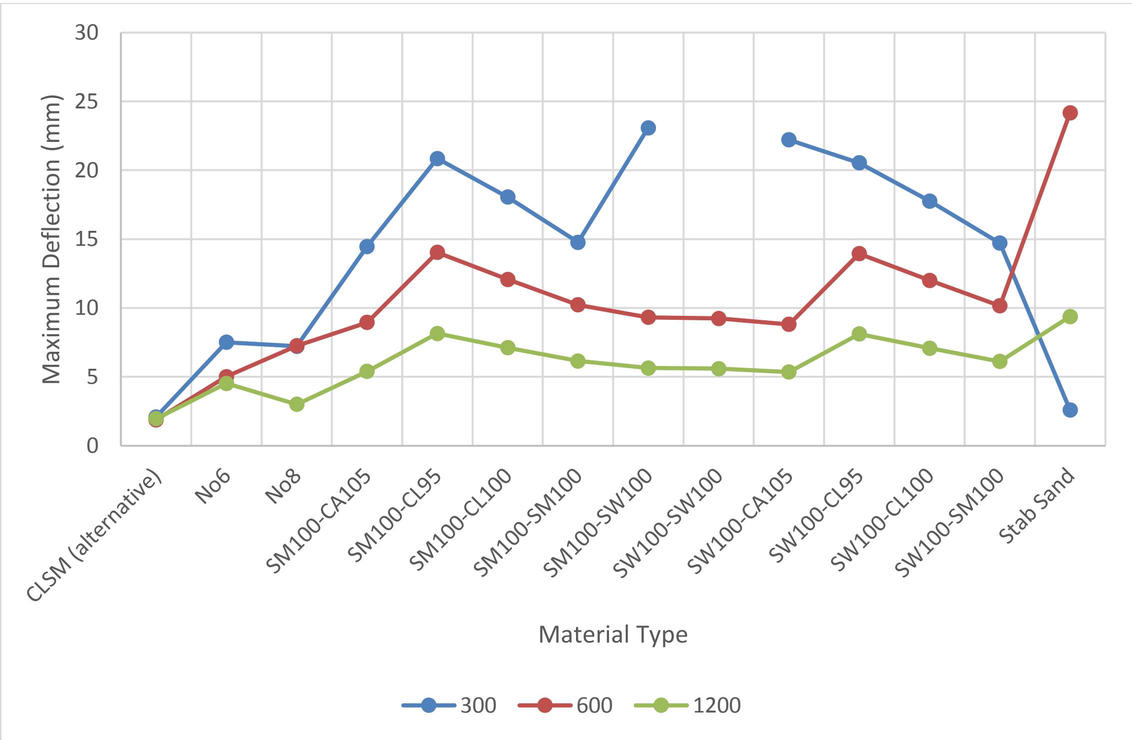


Figure 5.32: Maximum Deflection in the Y Direction

5.4.2 Soil Stress, Thrust, Shear and Moment

As shown by Figure 5.33 to Figure 5.36, the soil stress, thrust, shear and moment again follow the trend of maximum values for the 300 mm cover depth and minimum for the 1200 mm cover depth, except for the moment of the stabilised sand with the 600 mm moment being equal to the 300 mm depth. The maximum stress occurred in the stabilised sand at 414 kPa (300 mm cover) and the minimum stress was from CLSM at 28 kPa (1200 mm cover). Figure 5.34 indicated a maximum thrust from the stabilised sand of 39.9 kN/m and a minimum from all three cover depths of CLSM at 2.2 kN/m. Conversely the maximum shear occurred in the CLSM with a value of 42 kN/m (300 mm cover depth) and the minimum occurred in the No8 gravel with a value of 3.45 kN/m (1200 mm cover depth). Once again the minimum value for moment occurred in the CLSM at 0.04 kNm/m for the 1200 mm cover depth. The maximum value for moment occurred in the SW100-CL95 (300 mm cover) at 2.8 kNm/m

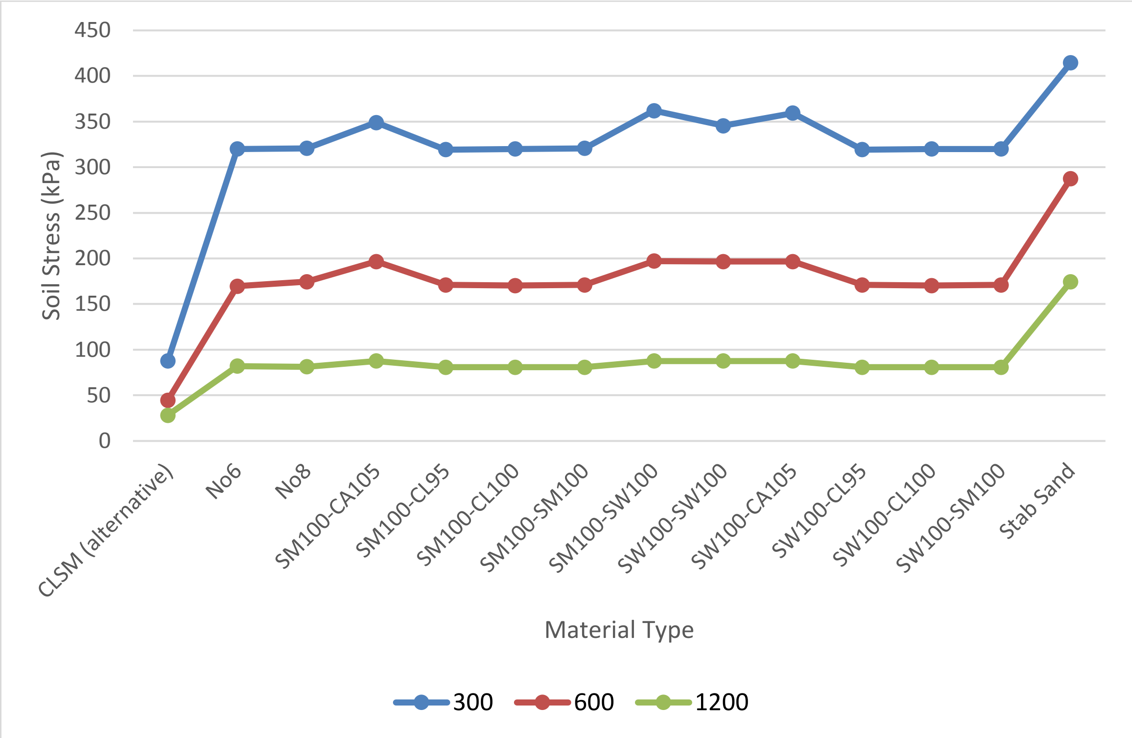


Figure 5.33: Maximum Soil Stress

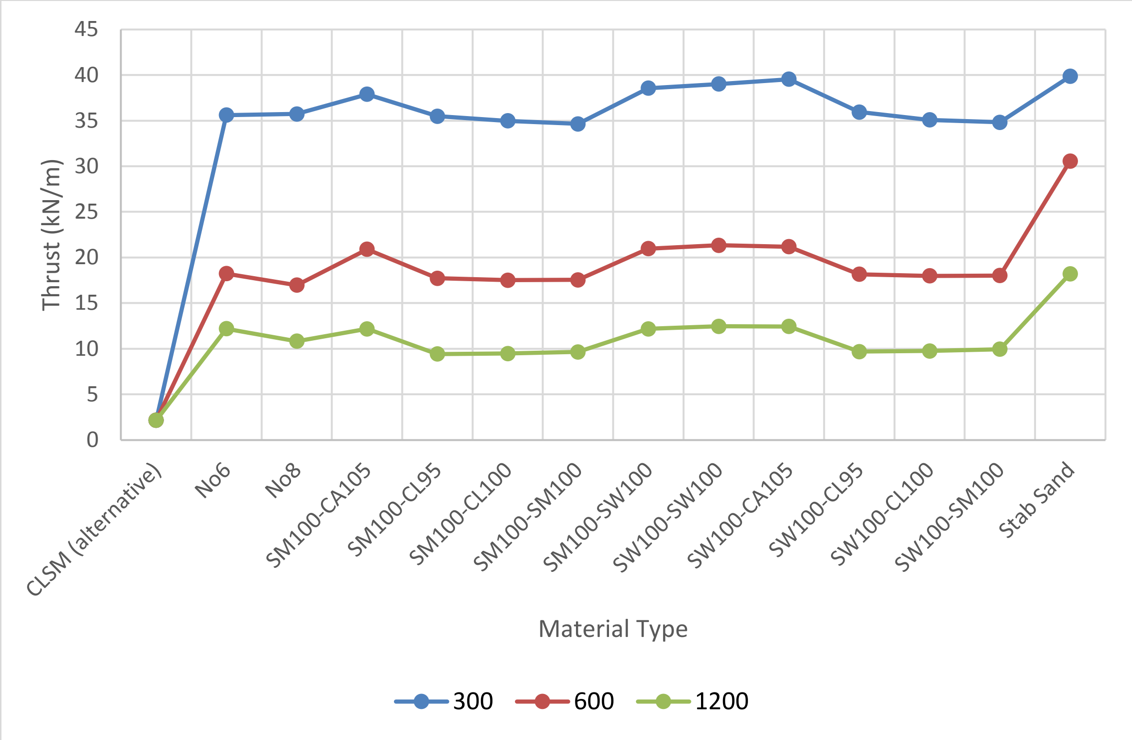


Figure 5.34: Maximum Thrust

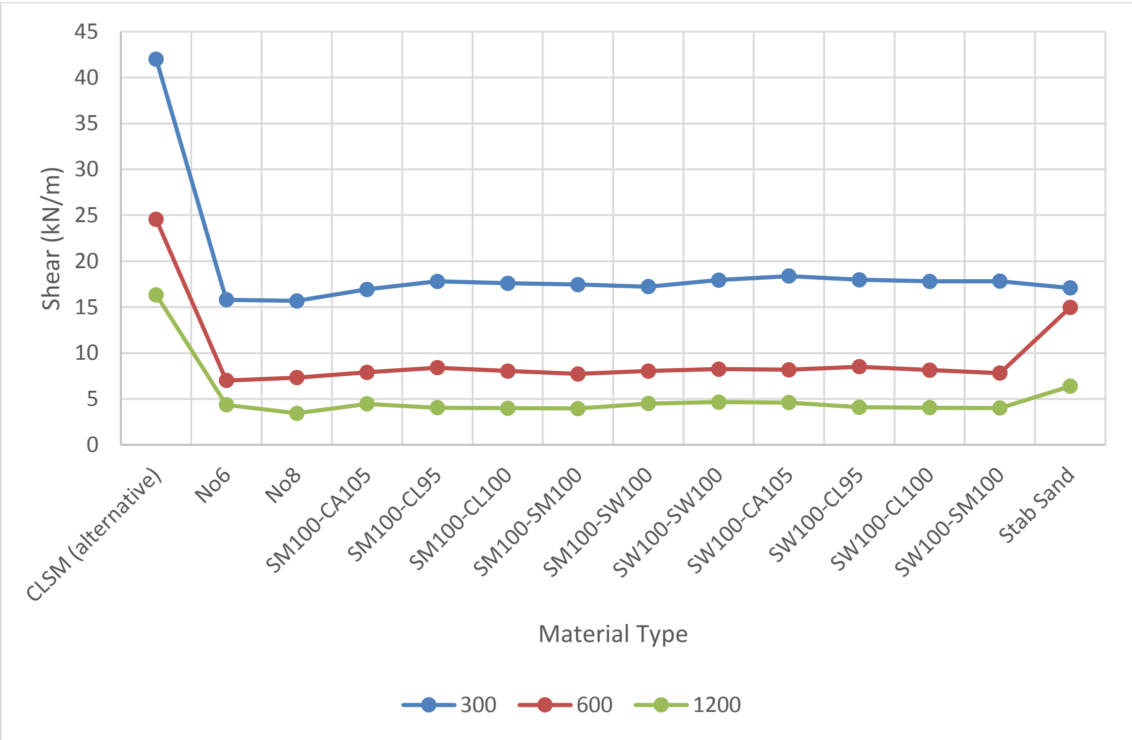


Figure 5.35: Maximum Shear

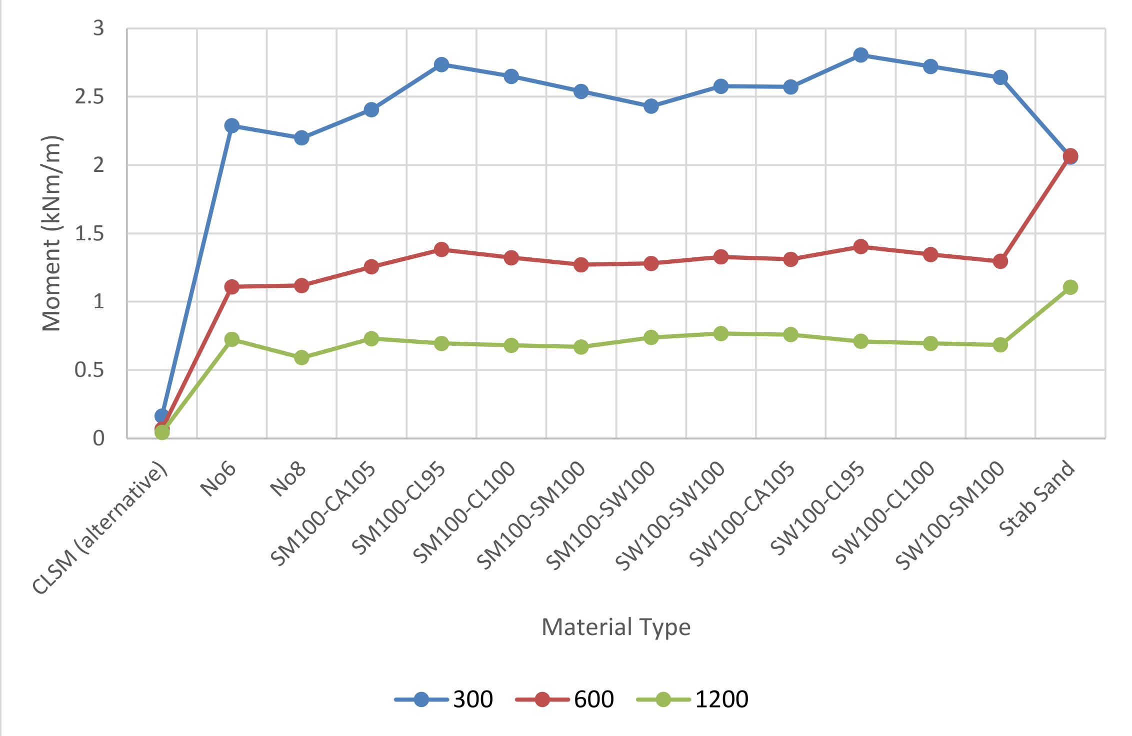


Figure 5.36: Maximum Moment

5.4.3 Safety Factors against Failure

Safety factors for steel yielding (Figure 5.37) from No8 (1200 mm) gave the maximum value of 14.0 and SW100-CL95 (300 mm) gave the minimum value of 2.6. This same trend occurred for concrete crushing and shear failure (Figure 5.38 and Figure 5.39) with No8 having the greatest values (Crushing = 11.7 and Shear = 26.1) and SW100-CL95 had the lowest values (Crushing = 2.17 and Shear = 4.75). The stabilised sand 300 mm layer had higher values than the 600 mm cover depth for each of the failure modes (except shear failure). The values for the CLSM alternative material were not included due to excessively high safety factors calculated within the model.

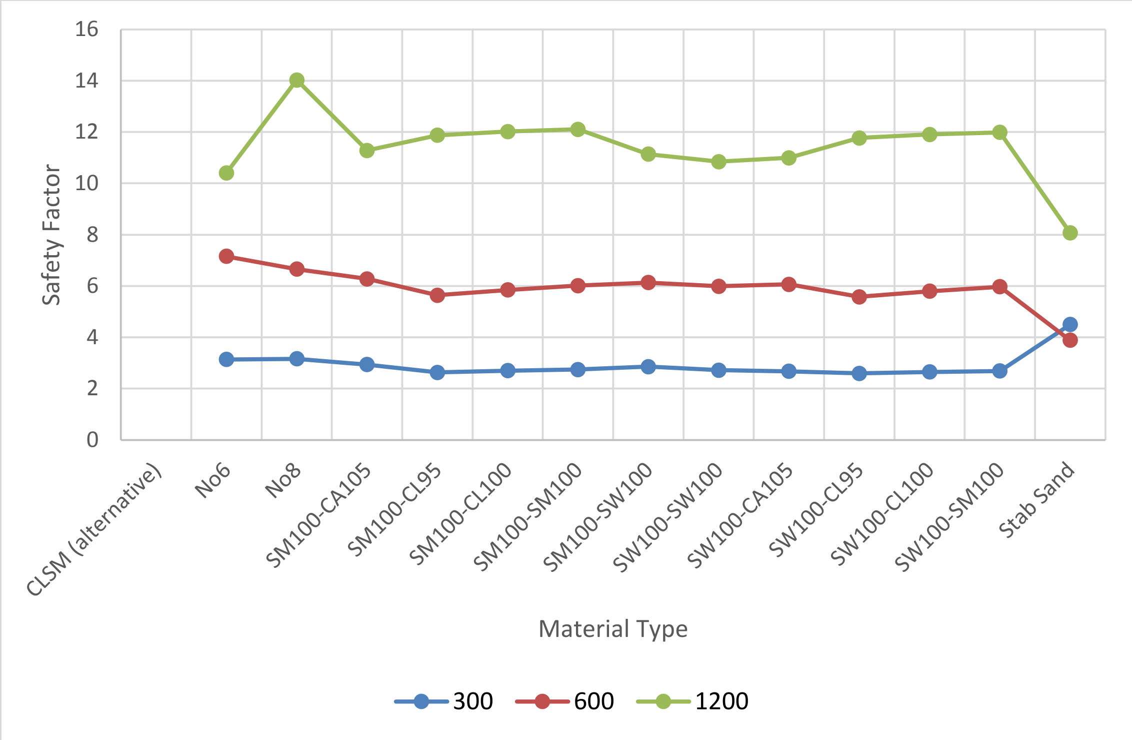


Figure 5.37: Safety Factor Steel Yielding

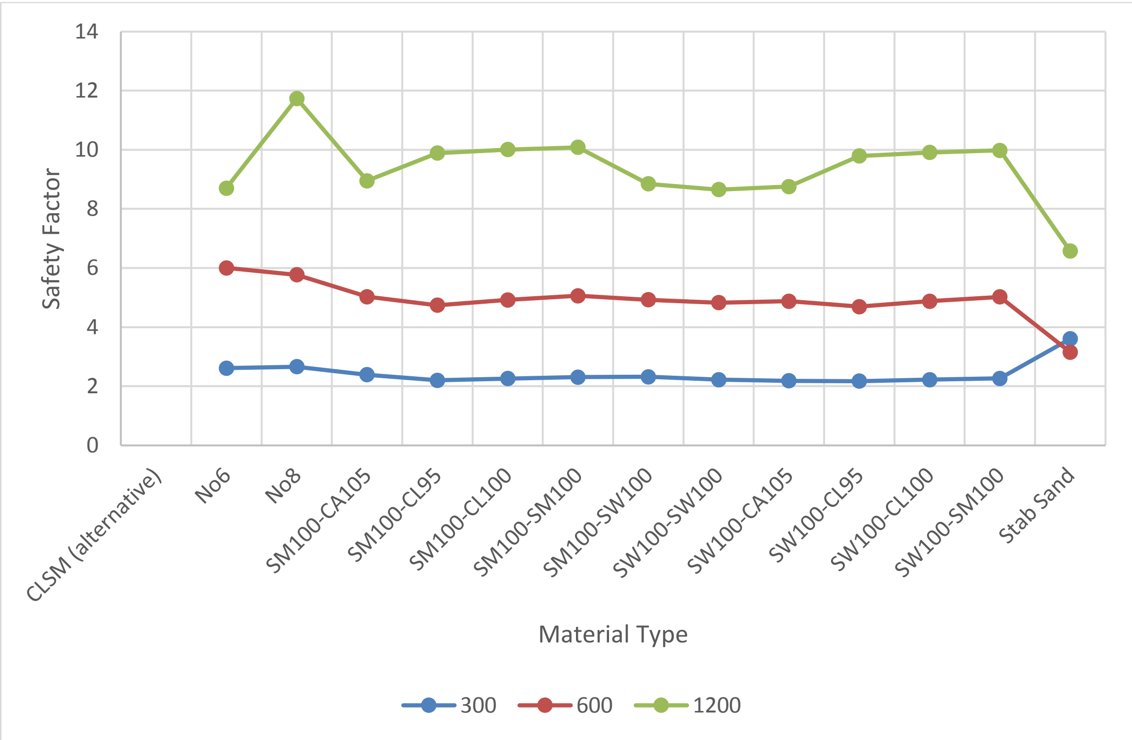


Figure 5.38: Safety Factor Concrete Crushing

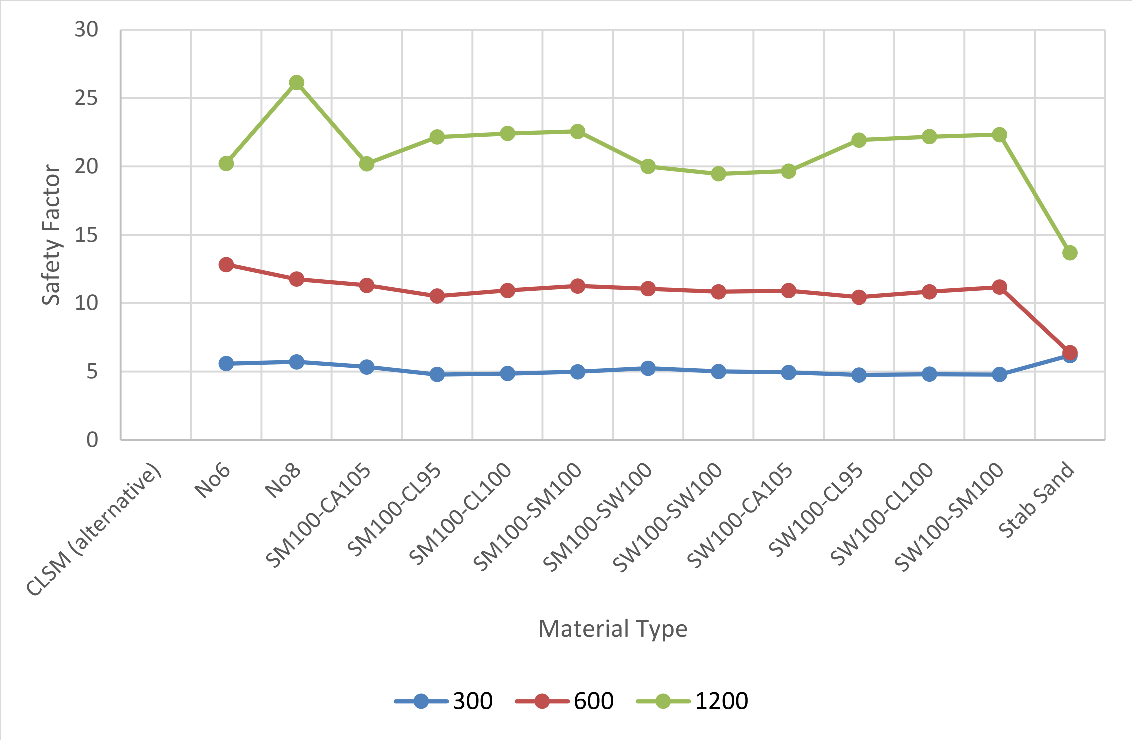


Figure 5.39: Safety Factor Shear Failure

5.5 AS/NZS 3725 Analysis

The analysis of the design scenarios was undertaken in accordance with Section 3.4 of this report. AS/NZS 3725 was also used to compare the results of the inventory data case studies. The results shown were not replicated for the SW100-Backfill materials as these would be considered the same as the SM100-Backfill material within the standard analysis method.

It was found that the conforming granular material had the most beneficial (greatest) bedding factors of 4.0 (dead load bedding factor) and 1.5 (live load bedding factor) (SM100-SW100, SW100-SM100, SM100-SM100 and SW100-SW100). This is based upon type HS3 support conditions as described in Committee:WS-006 (2007a). It is worth noting that the CLSM and stabilised sand could only be considered to provide type H support according to clause 9.2.2.1 of the standard and therefore had lower bedding factors (Committee:WS-006, 2007a). The applied live load was identical for all materials and the dead load only differed due to the unit weight of material. Hence, the material with the greatest unit weight had the greatest applied load prior to applying the relevant bedding factor.

The results demonstrated that the backfill providing 300 mm cover had safety factors less than one when the proof load was compared to the relevant design strength for a class IV 600 mm pipe culvert, from the Humes Pipe manual (Humes, 2009). The results indicated the SM100-CA105 and SW100-CA105 returned the lowest safety factor of 0.515 and the SM100-SM100 returned the greatest safety factor of 0.616. The results for the 600 mm and 1200 mm backfill covers had the same results for the performance of the backfill materials. The maximum and minimum safety factors for the 600 mm cover were 3.38 and 2.45 respectively. For the 1200 mm cover depths the maximum and minimum were 6.47 and 3.46 respectively (Figure 5.40). For further details refer to the results in Appendix B.

The analysis of the two case studies determined that case 1 with a cover depth of 400 mm would have safety factors of 2.45 and 1.97 for the conforming and aggregate backfill respectively. For case study 2 the cover depth was less than 400 mm, therefore, the standard required the culvert to be analysed with the load directly upon the pipe. This resulted in safety factors of 0.31 for both backfill cases (conforming and stabilised sand). The details of the analysis can be seen in Appendix B.

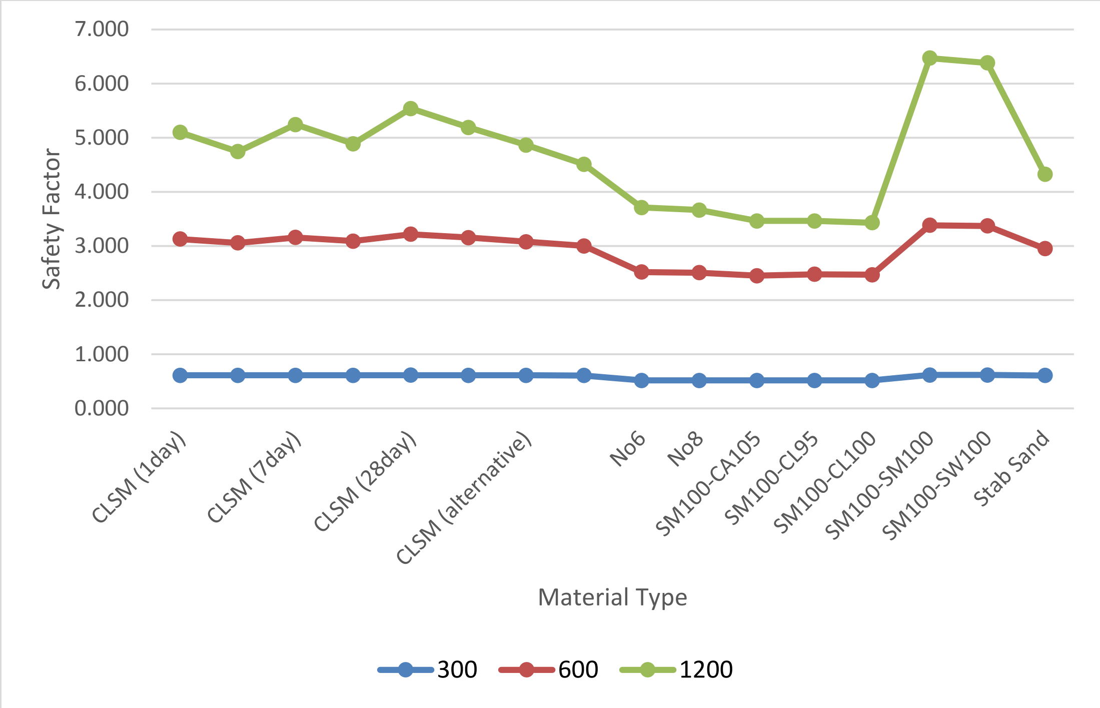


Figure 5.40: Safety Factor for AS/NZS 3725 Design

5.6 Model Comparison

A summary and comparison of the analysis methods above was undertaken, utilising node 56 (pipe crown), to compare the deflections, soil stress, thrust, shear and moment upon and directly above the pipe on average across the soil depths (refer to Appendix C for results). The factors of safety for each analysis method were also compared (Figure 5.9, Figure 5.10, Figure 5.28, Figure 5.29, Figure 5.38 and Figure 5.39). The safety factors for steel yielding were not compared as the Mohr Coulomb method did not determine steel yielding. The models were compared against the baseline Duncan Model as it was identified in the literature as best representing the real world data.

Trends between the data were not easily identifiable, however; it appears that the linear elastic model most closely matched the Duncan Model (Table 5.1). A comparison of the safety factors revealed that the Mohr Coulomb model was closer to the Duncan Model for lower cover depths and for the granular backfills with less cohesion. It is worth noting that there was minimal consistency between the material type and the proximity to the Duncan Model across different output types (i.e. deflection, shear stress, thrust etc.). Overall the safety factors against concrete crushing were relatively similar between the linear elastic and Mohr Coulomb models in comparison to the Duncan model (Table 5.2). However, the difference between the Mohr-Coulomb and Duncan model for shear failure was large, with the Mohr Coulomb model obtaining safety factors approximately three times that of the Duncan Model (Table 5.3).

Table 5.1: Comparison of Soil Models to the Duncan Model

Soil Model	Minimum % Difference	Maximum % Difference	Average % Difference
	300 mm Cover Depth		
Linear Elastic	0.0	89.9	13.0
Mohr Coulomb	0.3	57.1	13.1
	600 mm Cover Depth		
Linear Elastic	0.0	39.5	6.5
Mohr Coulomb	0.1	79.6	15.3
	1200 mm Cover Depth		
Linear Elastic	0.0	37.3	9.2
Mohr Coulomb	1.3	56.2	16.2

Table 5.2: Percent Difference - Safety Factor Concrete Crushing

Concrete Crushing	300 LE-D	300 MC-D	600 LE-D	600 MC-D	1200 LE-D	1200 MC-D
SM100-CA105	11.98	-13.61	3.30	1.30	-3.11	15.84
SM100-CL95	10.85	-16.51	6.55	-8.24	3.80	-6.11
SM100-CL100	9.27	-14.89	13.31	-17.48	15.16	-20.31
SM100-SM100	4.32	-7.55	0.44	-2.60	-8.70	6.09
SM100-SW100	4.62	-3.11	-0.92	6.26	-0.29	13.66
SW100-SW100	4.19	1.90	-3.80	9.93	-2.48	16.75
SW100-CA105	11.37	-3.36	1.68	5.06	-5.56	18.95
SW100-CL95	9.43	-13.13	7.68	-8.81	2.30	-3.98
SW100-CL100	9.24	-12.56	11.69	-15.92	14.08	-18.36
SW100-SM100	4.37	-5.69	-2.01	0.16	-10.51	7.97

Table 5.3: Percent Difference - Safety Factor Shear Failure

Shear Failure	300 LE-D	300 MC-D	600 LE-D	600 MC-D	1200 LE-D	1200 MC-D
SM100-CA105	5.51	-337.96	-1.53	-295.93	-10.80	-229.70
SM100-CL95	4.96	-337.97	1.42	-321.27	-3.35	-312.42
SM100-CL100	3.58	-329.06	7.64	-356.03	9.59	-371.45
SM100-SM100	-1.07	-305.95	-1.43	-318.71	-12.60	-278.44
SM100-SW100	-1.81	-304.16	-6.73	-272.66	-7.16	-240.64
SW100-SW100	-1.79	-282.57	-9.03	-260.57	-10.23	-224.80
SW100-CA105	4.67	-302.77	-3.59	-278.07	-13.92	-214.73
SW100-CL95	4.09	-330.58	1.23	-319.00	-4.82	-304.97
SW100-CL100	3.48	-322.78	6.12	-350.15	8.20	-362.58
SW100-SM100	-26.92	-290.46	-4.10	-306.85	-14.65	-270.41

A comparison has been made of the limiting safety factors of the FEM models to the AS/NZS 3725 safety factor in Table 5.4, Table 5.5 and Table 5.6. The results highlight the low safety factor identified within the 300 mm cover depth by the AS/NZS 3725 method. It also shows that the AS/NZS 3725 calculates a greater safety factor when comparing the conforming granular materials (SW100 and SM100) to the FEM model results. It is clear that the standard also calculates reduced safety factors for non-conforming materials even when the FEM models report improved performance.

Table 5.4: Comparison of Limiting Safety Factors for the 300 mm Cover Depth

	LE	D	MC	AS/NZS 3725
CLSM (1day)	2.69	NA	NA	0.61
CLSM (7day)	2.93	NA	NA	0.61
CLSM (28day)	6.52	NA	NA	0.61
CLSM (alternative)	3.57	NA	NA	0.61
No6	1.43	NA	2.61	0.52
No8	1.44	NA	2.66	0.52
SM100-CA105	1.29	1.22	2.39	0.51
SM100-CL95	1.15	1.09	2.20	0.52
SM100-CL100	1.17	1.13	2.26	0.52
SM100-SM100	1.21	1.23	2.31	0.62
SM100-SW100	1.27	1.30	2.32	0.62
SW100-SW100	1.29	1.31	2.22	0.62
SW100-CA105	1.29	1.23	2.18	0.51
SW100-CL95	1.15	1.10	2.17	0.52
SW100-CL100	1.18	1.14	2.22	0.52
SW100-SM100	1.21	1.23	2.27	0.62
Stab Sand	6.87	NA	3.60	0.61

Table 5.5: Comparison of Limiting Safety Factors for the 600 mm Cover Depth

	LE	D	MC	AS/NZS 3725
CLSM (1day)	6.29	NA	NA	3.06
CLSM (7day)	5.12	NA	NA	3.09
CLSM (28day)	14.35	NA	NA	3.15
CLSM (alternative)	8.47	NA	NA	3.00
No6	3.12	NA	6.00	2.52
No8	3.16	NA	5.77	2.51
SM100-CA105	2.81	2.85	5.03	2.45
SM100-CL95	2.53	2.50	4.74	2.48
SM100-CL100	2.59	2.40	4.92	2.47
SM100-SM100	2.65	2.69	5.06	3.38
SM100-SW100	2.78	2.97	4.92	3.37
SW100-SW100	2.76	3.01	4.83	3.37
SW100-CA105	2.79	2.89	4.88	2.45
SW100-CL95	2.52	2.49	4.69	2.48
SW100-CL100	2.56	2.41	4.88	2.47
SW100-SM100	2.64	2.75	5.02	3.38
Stab Sand	18.47	NA	3.15	31.21

Table 5.6: Comparison of Limiting Safety Factors for the 1200 mm Cover Depth

	LE	D	MC	AS/NZS 3725
CLSM (1day)	7.80	NA	NA	4.74
CLSM (7day)	8.29	NA	NA	4.89
CLSM (28day)	12.99	NA	NA	5.19
CLSM (alternative)	11.69	NA	NA	4.51
No6	6.16	NA	8.70	3.71
No8	6.20	NA	11.74	3.66
SM100-CA105	5.53	6.12	8.95	3.46
SM100-CL95	5.20	5.37	9.89	3.46
SM100-CL100	5.26	4.75	10.01	3.43
SM100-SM100	5.29	5.96	10.08	6.47
SM100-SW100	5.48	5.87	8.85	6.38
SW100-SW100	5.43	5.99	8.65	6.38
SW100-CA105	5.48	6.25	8.76	3.46
SW100-CL95	5.16	5.41	9.79	3.46
SW100-CL100	5.22	4.79	9.91	3.43
SW100-SM100	5.26	6.03	9.98	6.47
Stab Sand	1.00	NA	6.58	0.00

5.7 Inventory Asset Analysis

5.7.1 Culvert Case 1

The analysis method significantly affected the results presented for culvert case 1. The scenario was analysed across the three different analysis methods with a conforming gravel compared to the No6 aggregate (noting that the Duncan Model does not have parameters for the aggregate). From this, the linear elastic analysis saw smaller surface deflections and smaller maximum X and Y deflections in the No6 compared to the conforming gravel (SM100-SM100), however, the Mohr Coulomb model saw the opposite. The Duncan model also seemed to give values in between the two other models for the conforming gravel (Figure 5.41, Figure 5.42 and Figure 5.43).

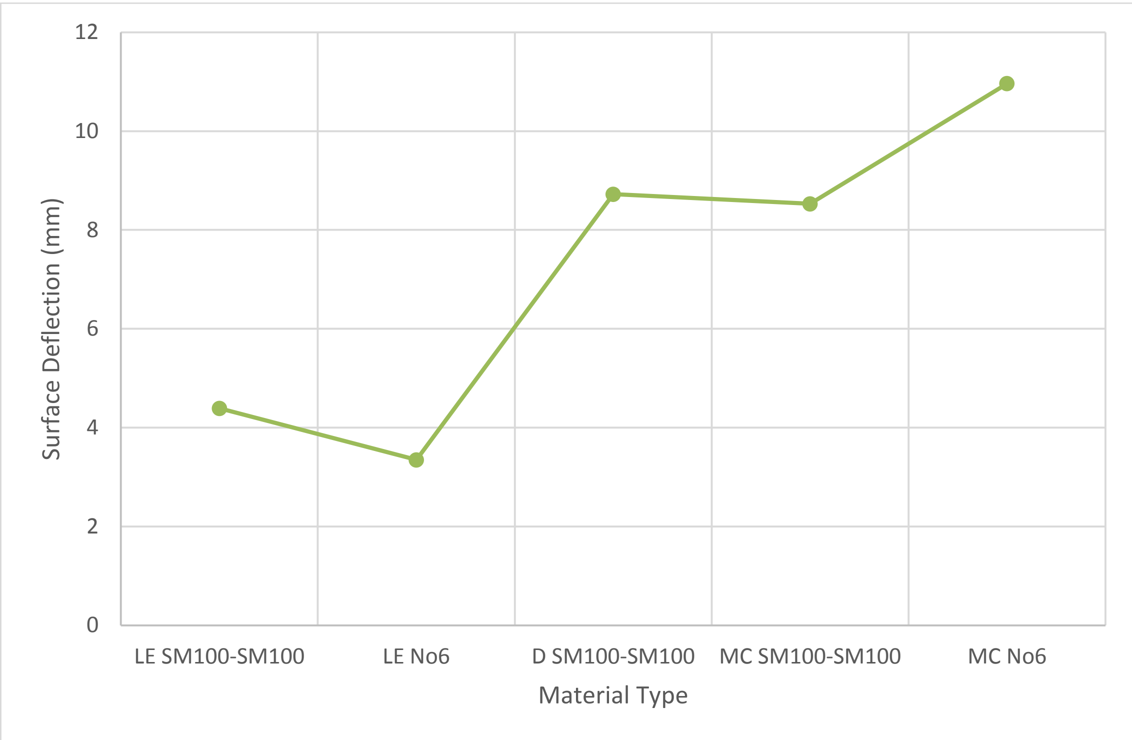


Figure 5.41: Surface Deflection

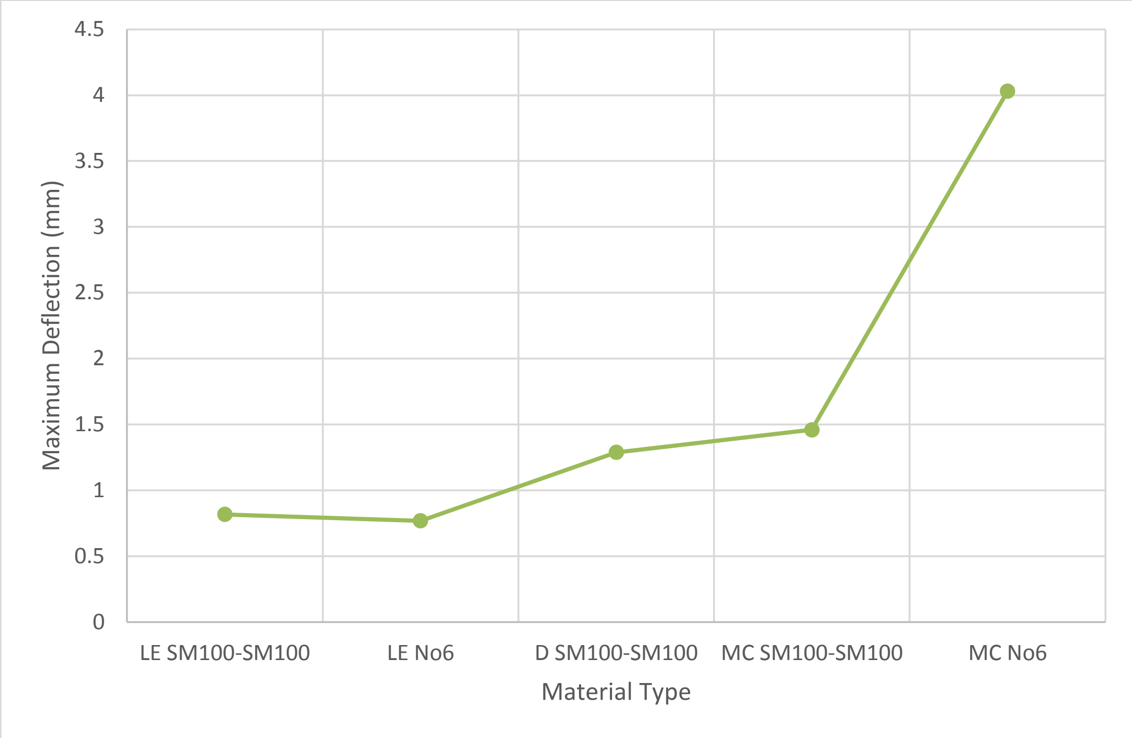


Figure 5.42: Maximum Deflection in the X Direction

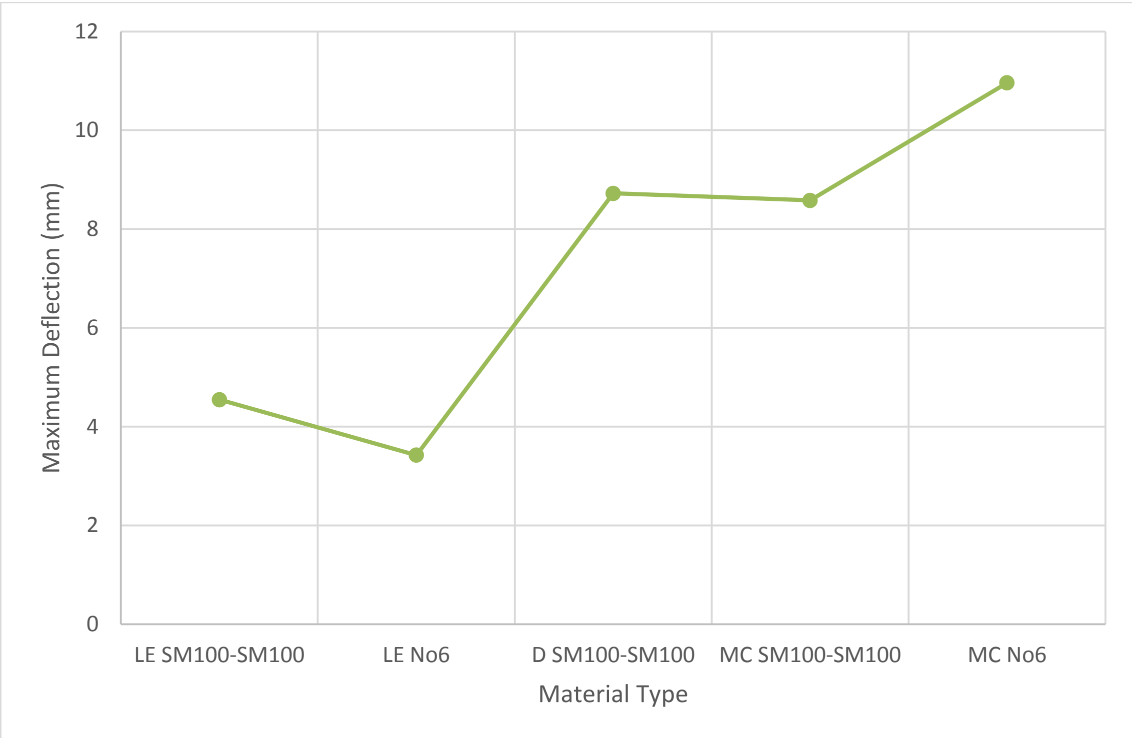


Figure 5.43: Maximum Deflection in the Y Direction

The maximum soil stress (Figure 5.44) was achieved in the linear elastic model for the conforming gravel while the No6 had a lower value. Once again the Mohr Coulomb model behaved the opposite to the linear elastic method between backfill types.

Figure 5.44: Maximum Soil Stress

For the maximum thrust and shear (Figure 5.45 and Figure 5.46), the models once again showed that there was no clear trend across models. Thrust and shear increased from conforming gravel to the aggregate in the linear elastic method while decreased in the Mohr Coulomb method.

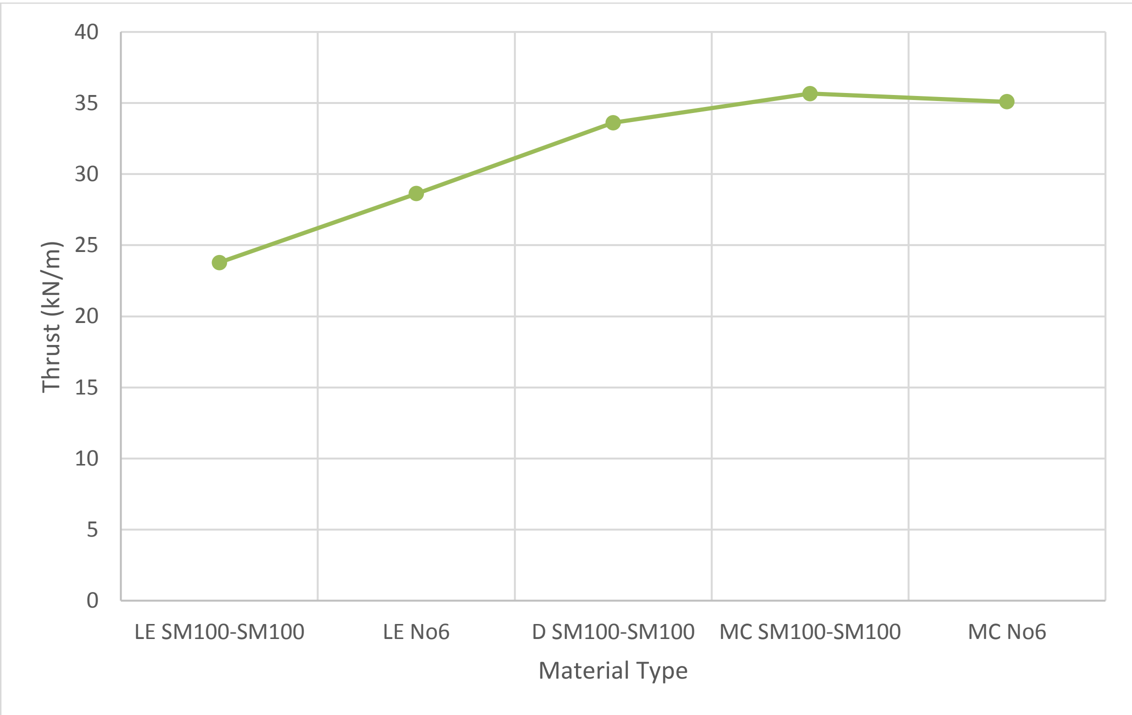


Figure 5.45: Maximum Thrust

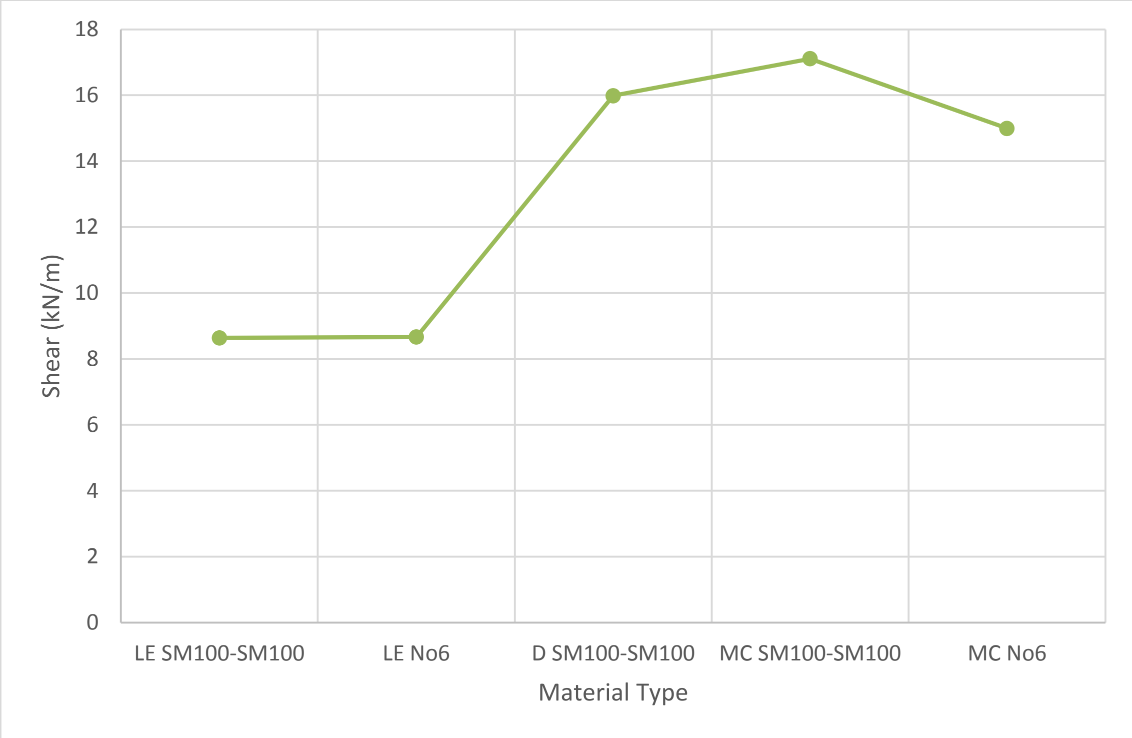


Figure 5.46: Maximum Shear

The maximum moment (Figure 5.47) dropped in both models from the conforming gravel to the non-conforming material and once again the Duncan Model obtained a value approximately in between the other two models.

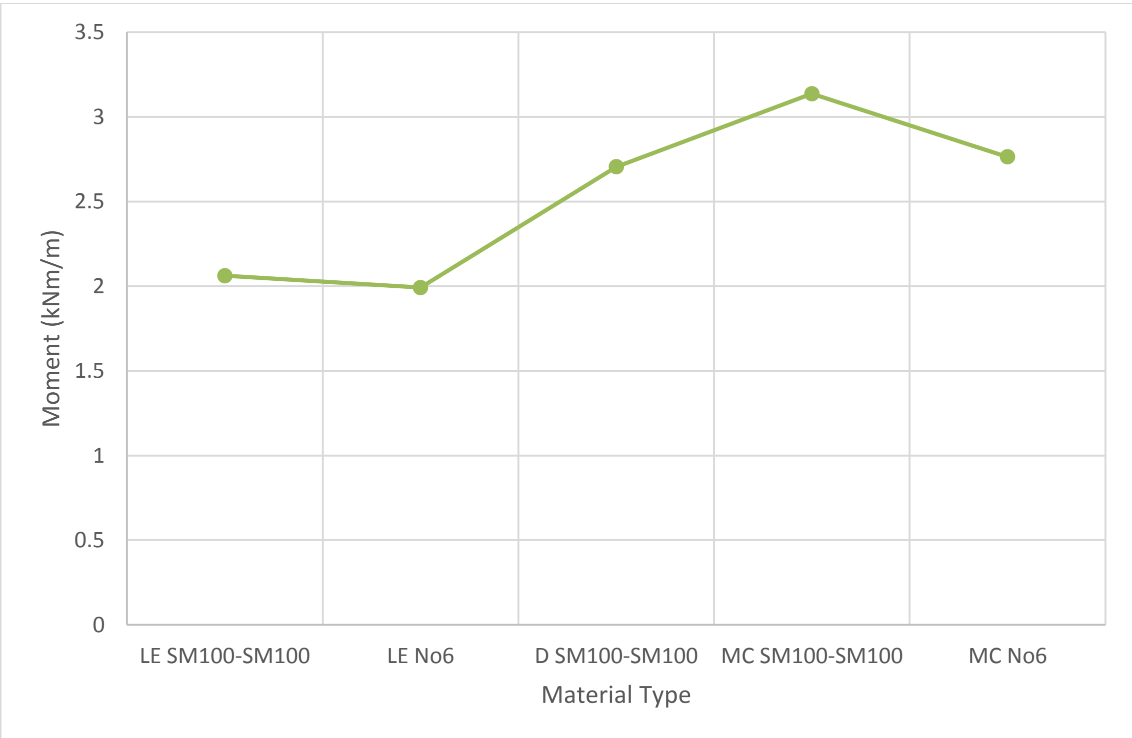


Figure 5.47: Maximum Moment

The safety factors against failure (Figure 5.48 to Figure 5.50) indicated that the aggregate performed slightly better than the conforming gravel in all cases except the linear elastic shear failure where the values were approximately the same. The greatest safety factor against concrete crushing varied from 2.39 for the granular backfill modelled using the Mohr Coulomb method to a maximum of 4.5 for the aggregate backfill modelled using a linear elastic model. The opposite was evident for safety factors against shear failure with a minimum from the linear elastic aggregate backfill (3.2) and a maximum for the Mohr Coulomb aggregate backfill (8.2). Also note that the values for steel yielding for the Duncan model were identified as 1000 and therefore not included in the results.

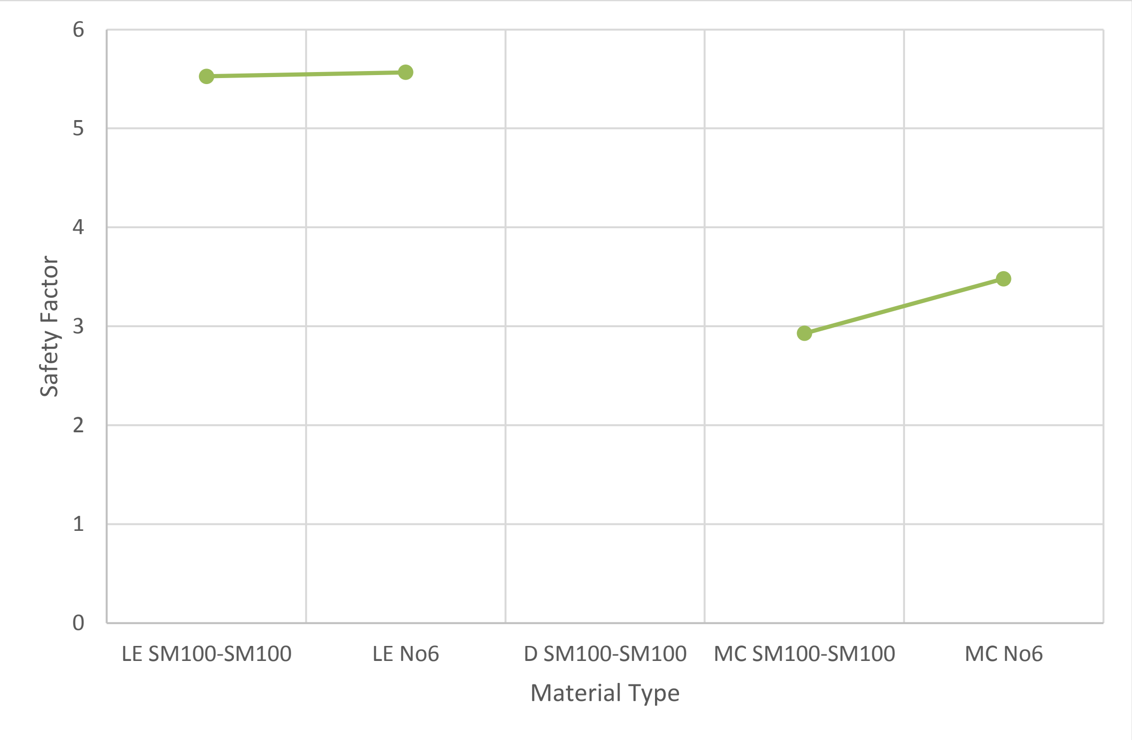


Figure 5.48: Safety Factor for Steel Yielding

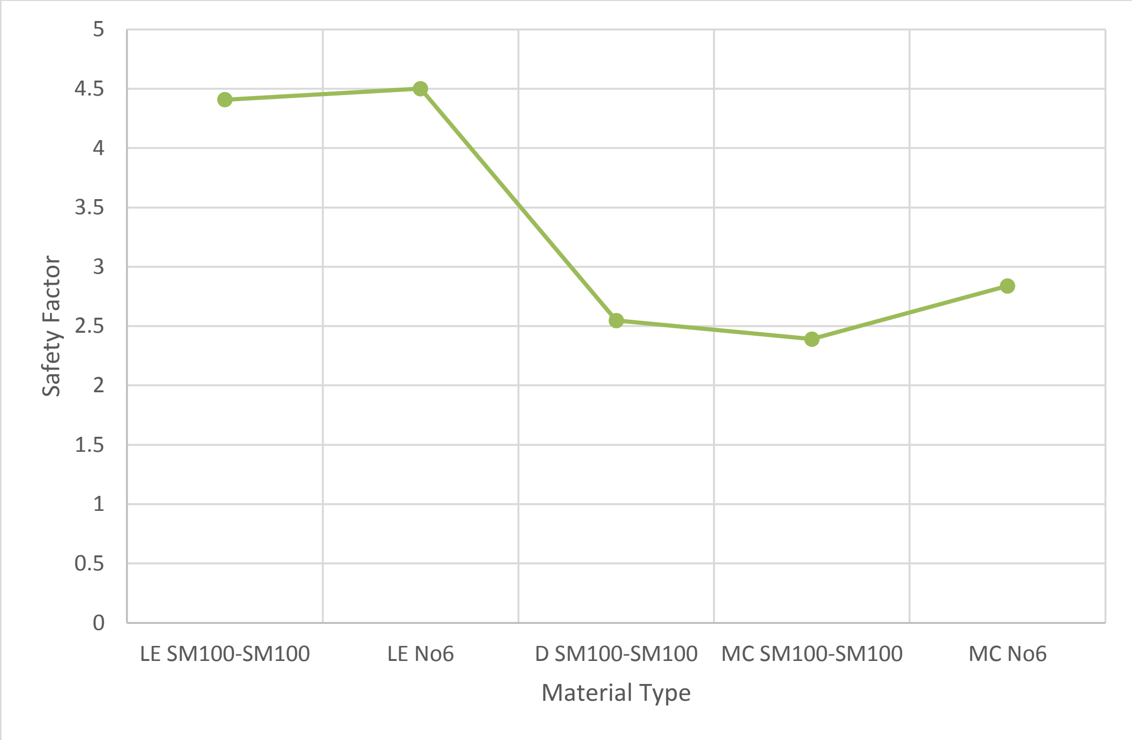


Figure 5.49: Safety Factor Concrete Crushing

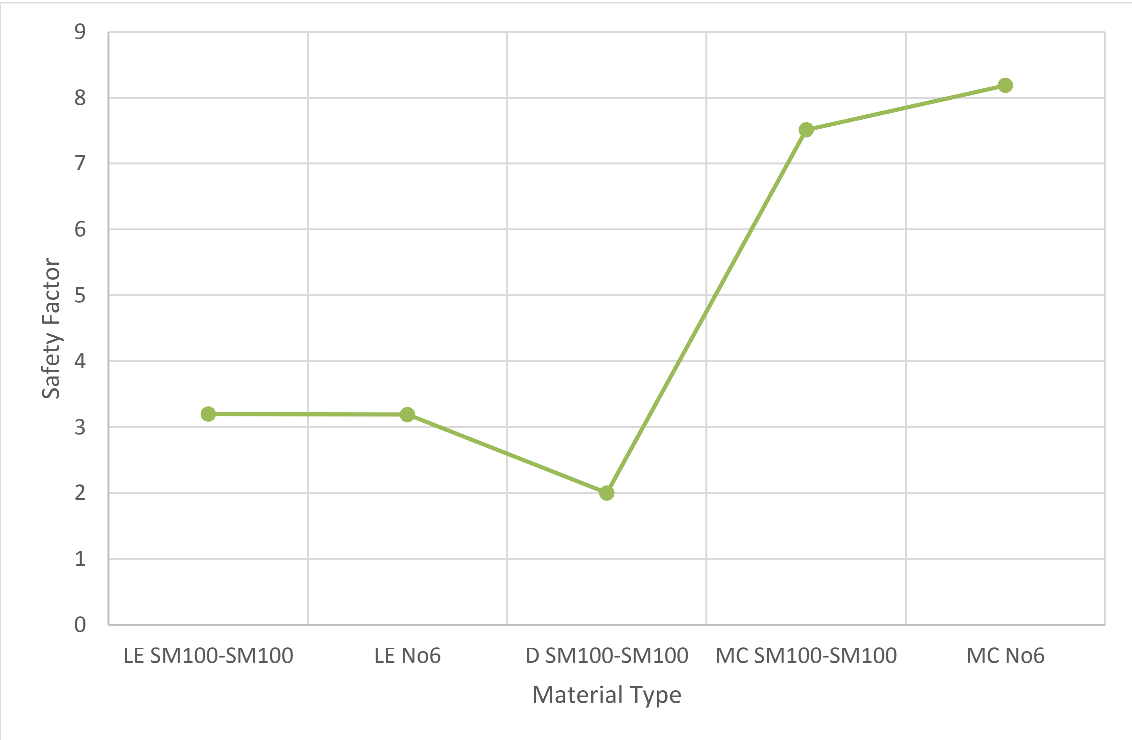


Figure 5.50: Safety Factor Shear Failure

5.7.2 Culvert Case 2

Culvert case 2 was also analysed utilising the three different models, with the Duncan model once again not having material parameters for the stabilised sand alternative backfill. The values for surface, X and Y deflection obtained from the Mohr Coulomb model far exceeded the linear elastic results. In addition, the behaviour between materials did not follow the same trends (for example the surface deflection decreased between the linear elastic granular material to the stabilised sand but increased from the Mohr Coulomb granular material to the stabilised sand). As identified in Culvert Case 1, the Duncan models' conforming material behaviour was located between the linear elastic and Mohr Coulomb method (Figure 5.51, Figure 5.52 and Figure 5.53).

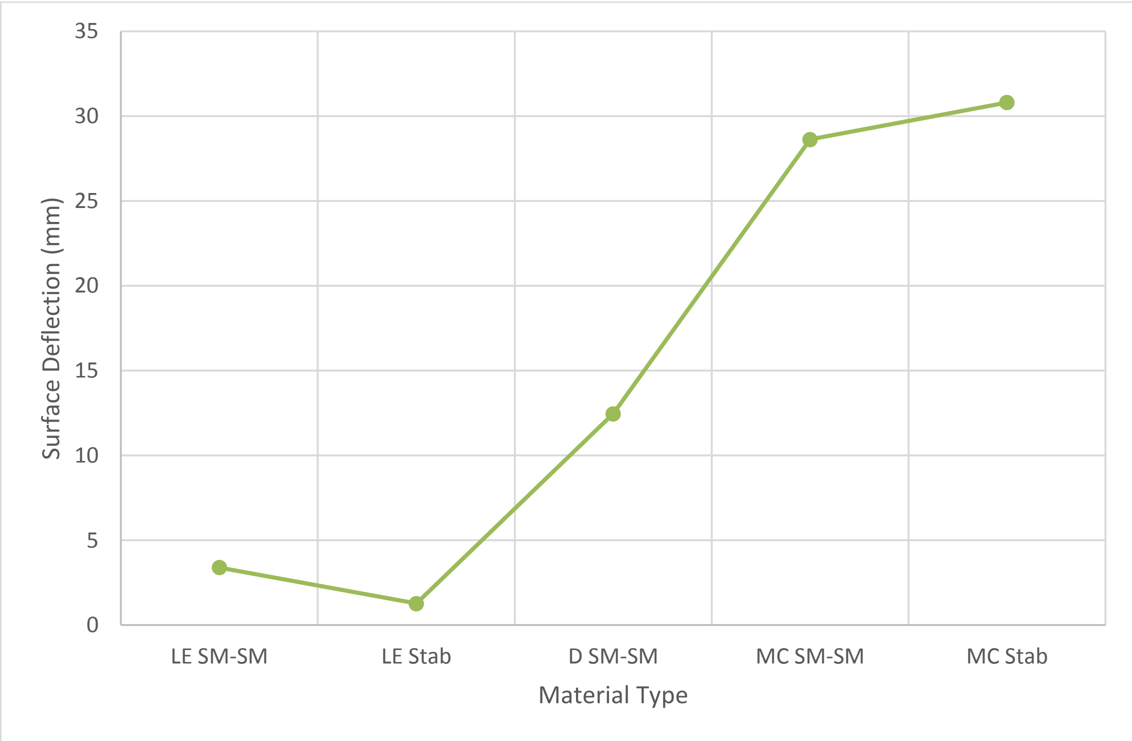


Figure 5.51: Deflection at the Surface

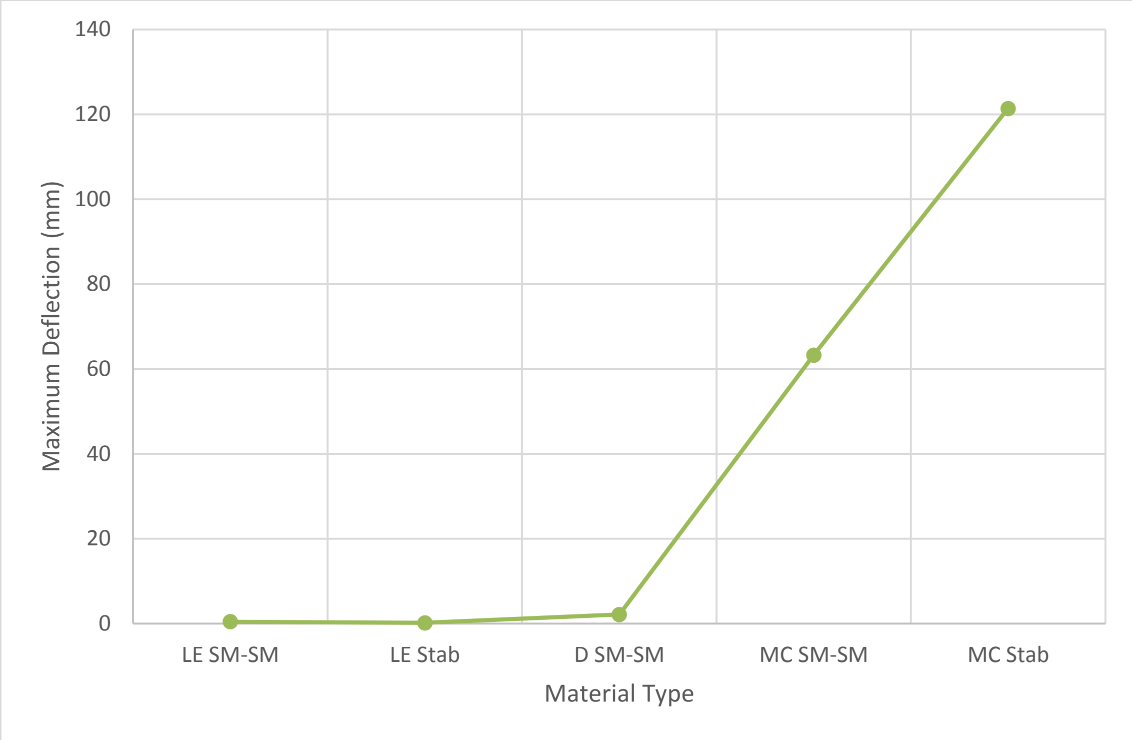


Figure 5.52: Maximum Deflection in the X Direction

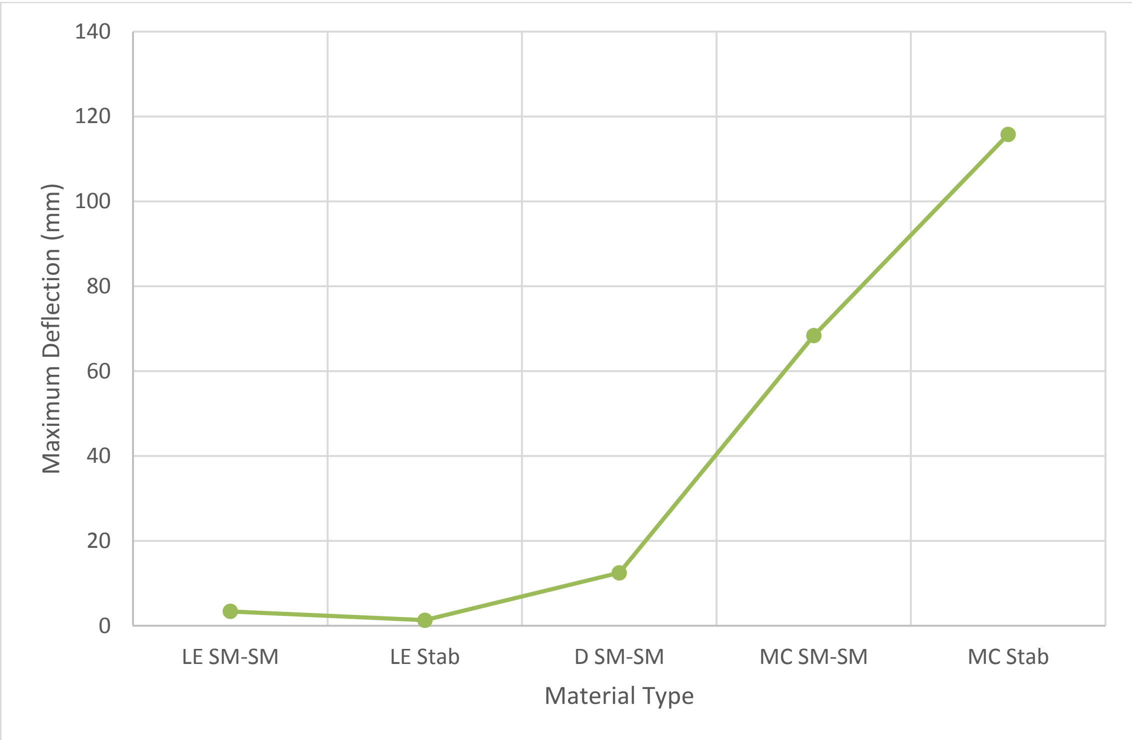


Figure 5.53: Maximum Deflection in the Y Direction

The maximum soil stress and thrust also had the opposite behaviour between the two materials across the two models. The highest stress was found in the SM100-SM100 material analysed using the linear elastic method. The largest thrust was achieved by the stabilised sand using linear elastic analysis (Figure 5.54 and Figure 5.55).

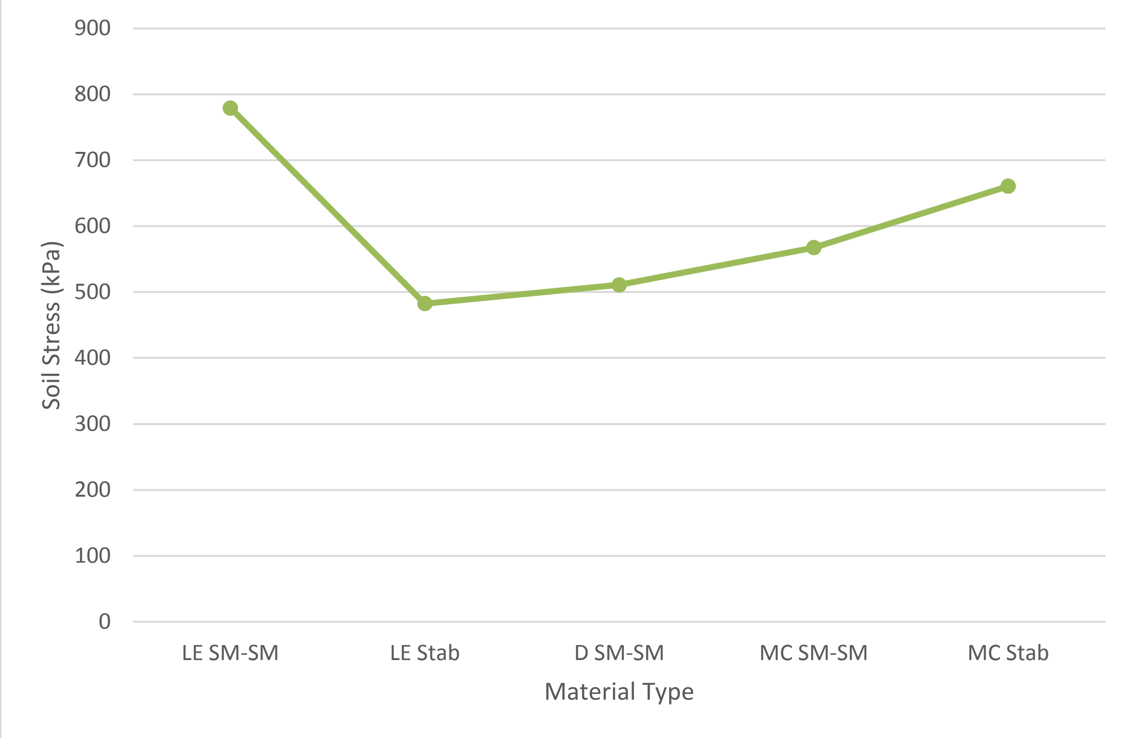


Figure 5.54: Maximum Soil Stress

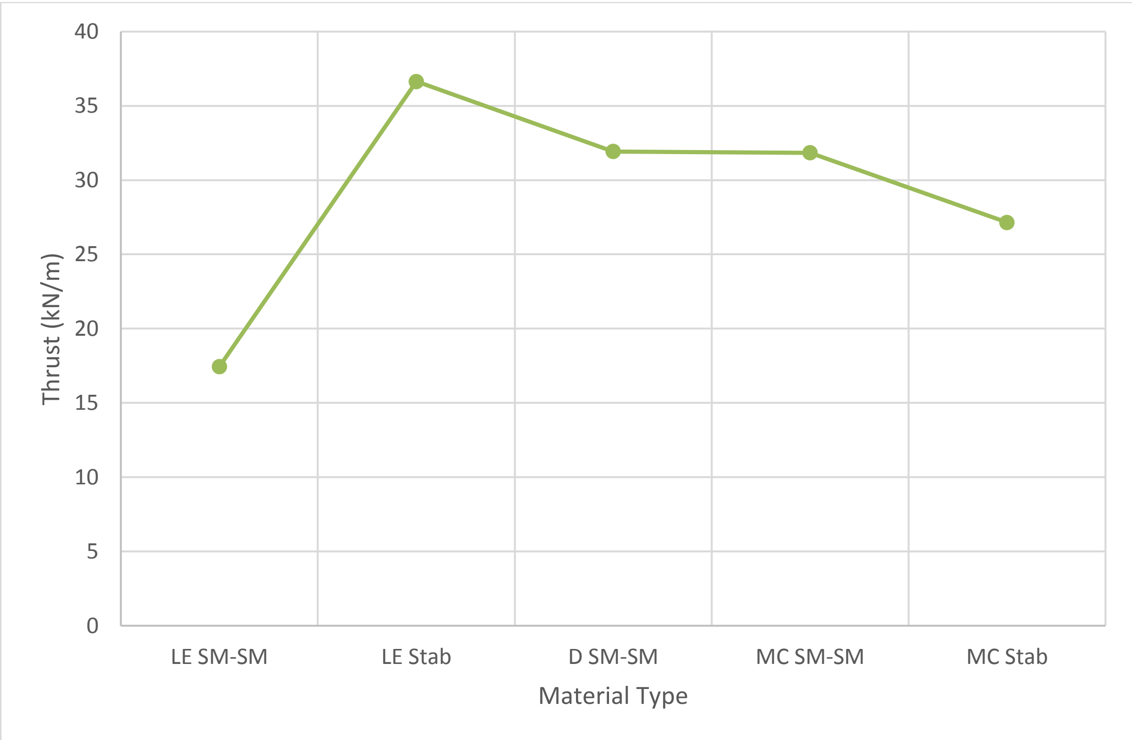


Figure 5.55: Maximum Thrust

The maximum shear and moment showed similar trends across the soil models, however, the results were quite different. This can be seen in Figure 5.56 and Figure 5.57 where the maximum shear and moment for the stabilised sand were 3.2 kN/m and 0.22 kNm/m for linear elastic analysis and 11.4 kN/m and 0.72 kNm/m for the Mohr Coulomb analysis.

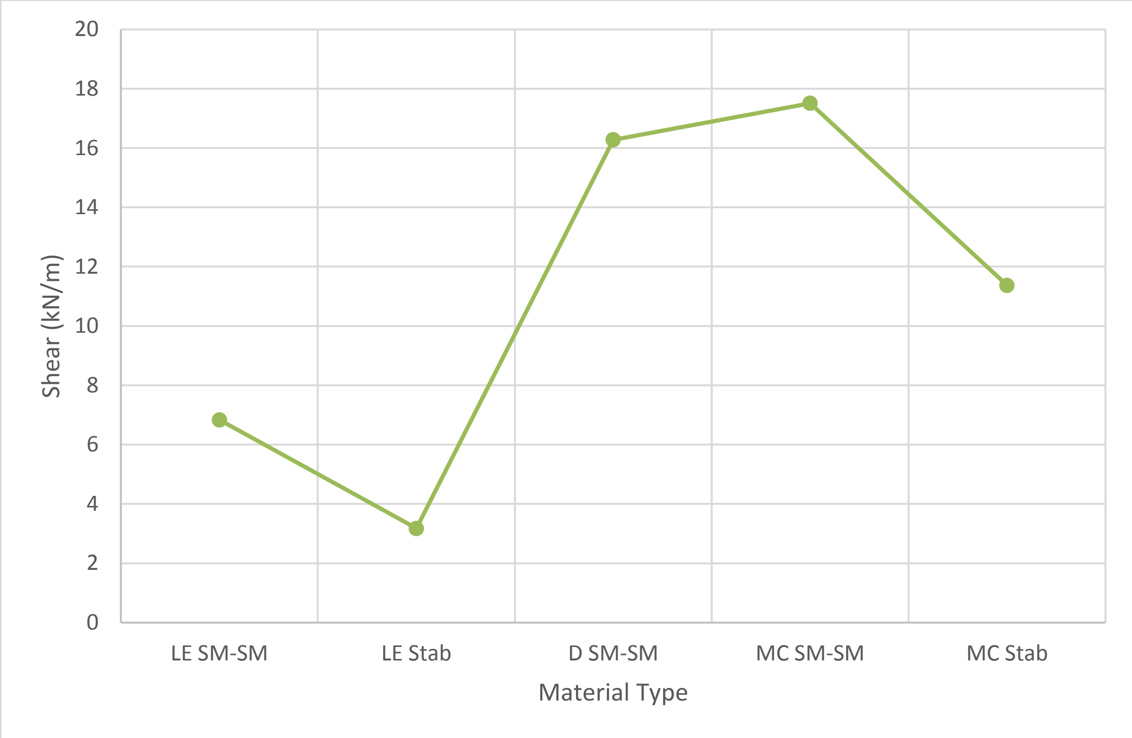


Figure 5.56: Maximum Shear

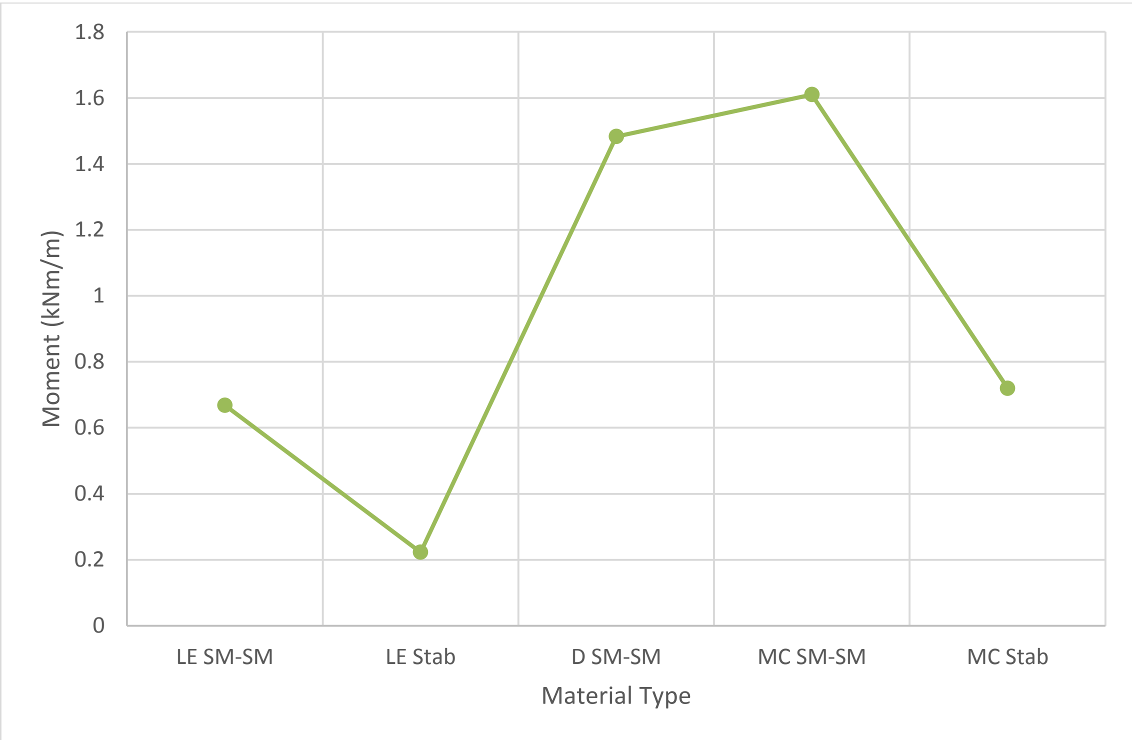


Figure 5.57: Maximum Moment

Once again the Duncan model result for steel yielding was not included as it was an outlier however, the linear elastic and Mohr Coulomb results followed the same trend between materials (increasing from the granular material to the stabilised sand) (Figure 5.58). The factors of safety also followed the same trends for crushing and shear failure with the safety factor increasing from the conforming gravel to the stabilised sand (Figure 5.59 and Figure 5.60). The limiting safety factor was for shear failure with the Duncan model predicting failure (safety factor =0.69) for the conforming gravel, which is what occurred in the field. Once again the factor of safety against steel yielding was quite high.

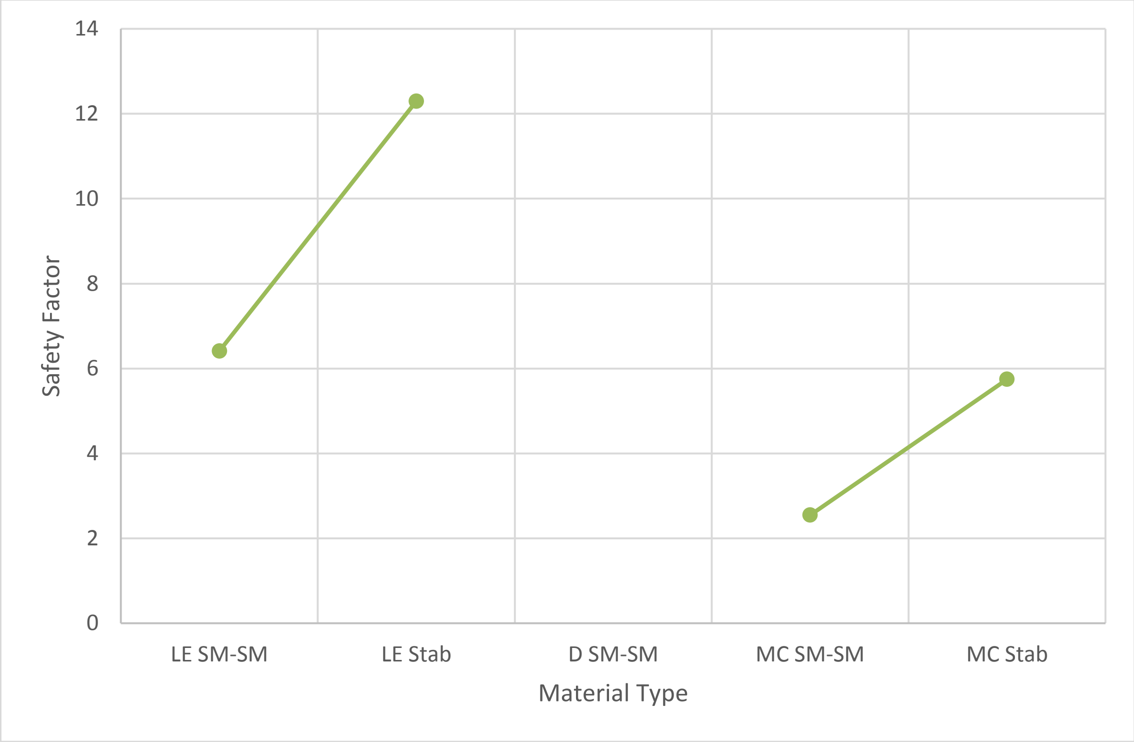


Figure 5.58: Safety Factor Steel Yielding

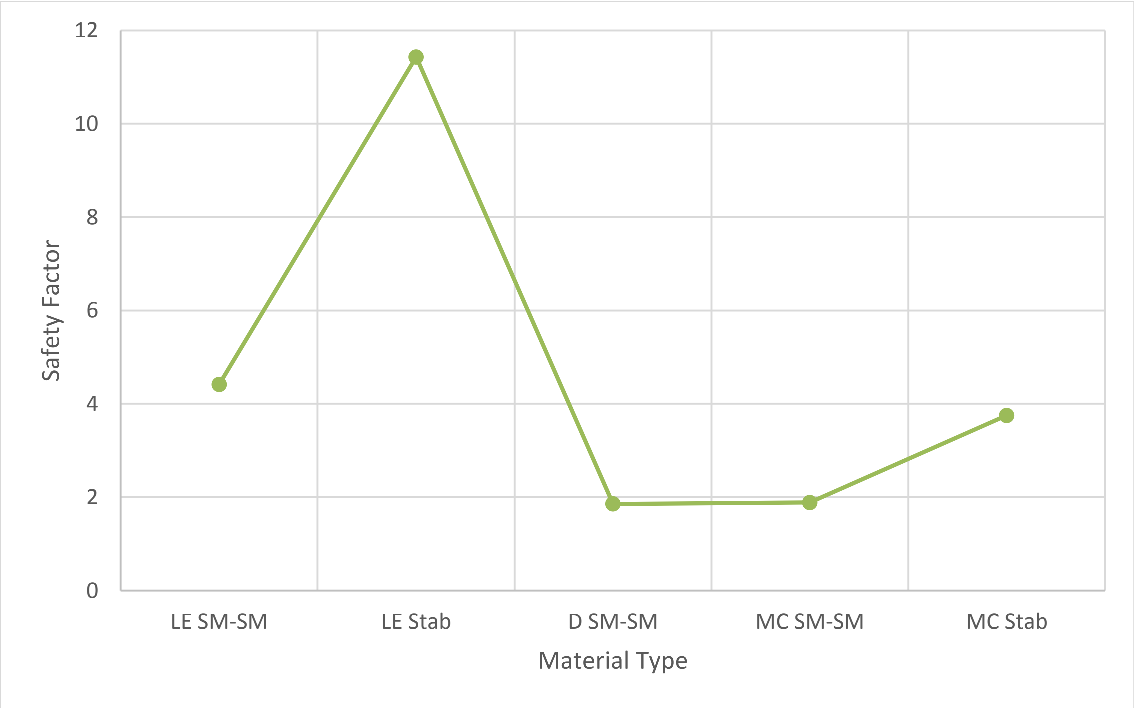


Figure 5.59: Safety Factor Concrete Crushing

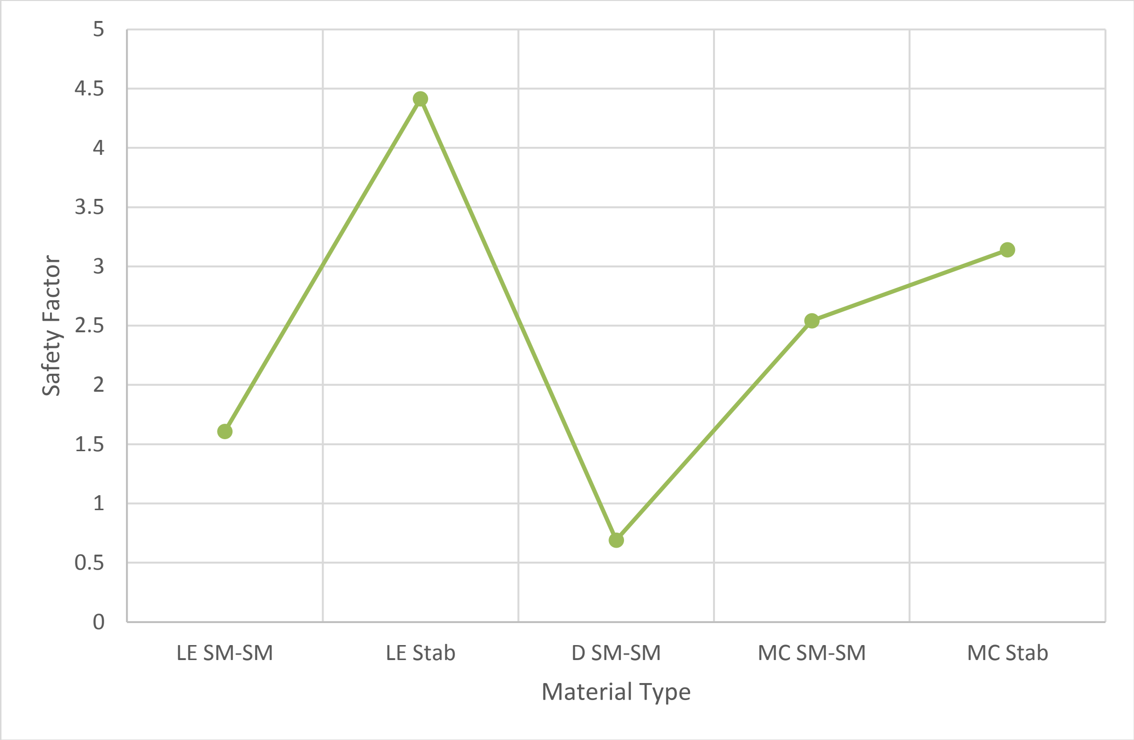


Figure 5.60: Safety Factor Shear Failure

6 Discussion

6.1 Model Comparison

A comparison has been made between the current Australian standard, a simplified linear elastic model, the Duncan model and the Mohr Coulomb model. The model comparison has been undertaken to directly assess the differences between various analysis techniques for calculating the load distribution and soil movement surrounding pipe culverts. The articles by NCHRP (2010) and Kitane and McGrath (2006) both indicate that the Mohr Coulomb model and a hardening model returned similar results, while the linear elastic model is noted as being "significantly different" (NCHRP, 2010). In most cases the results returned within the various scenarios of this project did not match the above conclusions. It was found that on average the linear elastic result more closely matched the Duncan model in comparison to the Mohr Coulomb method. Although, the Mohr Coulomb method did tend to perform better for lower cover depths. This is proposed to be due to the loading conditions under which these scenarios have been exposed. With lower cover depths the live load was greater leading to a higher potential for plastic failure. As such, the Mohr Coulomb model is more likely to undergo plastic flow. At these cover depths the Duncan Model may also experience shear failure which may lead to the models returning close results. The Duncan and Mohr Coulomb models do not share the same soil failure condition, although it is similar. Therefore, for higher cover depths or for scenarios where the shear stress may just exceed the maximum shear there is still the potential for the linear elastic model to more closely match the Duncan Model (Katona, 2015). This is due to the initial hardening of the Duncan model limiting the effect of the shear failure that develops at the final loading stage. It is also worth noting that the relationship referred to by NCHRP (2010) and Kitane and McGrath (2006) does not specifically refer to the Duncan model but rather a hardening model. NCHRP (2010) specified a hardening model similar to the Duncan-Selig model and Kitane and McGrath (2006) specified the PLAXIS Hardening model. The Duncan model is generally considered as an accurate method for computing loads on culvert structures and as such has been utilised to develop codes and standards such as the AASHTO design procedure (Katona, 2015; Kitane & McGrath, 2006).

The models also showed minor variations in general, with the linear elastic model having a maximum average deviation from the Duncan model of 13.0% and a maximum average deviation of 16.2% from the Mohr Coulomb model. The variation in safety factor against concrete crushing was also less than 18% in all materials and backfill depths, excluding two. These two cases were in the 1200 mm cover depth comparing the Duncan Model to the Mohr

Coulomb model. However, it does not appear to specifically be the increase in cover depth that caused the increase in variation. The data indicates that the material type had more of an effect on the consistency between the models, with the Mohr Coulomb model deviating from the Duncan model for the SM100-CL100 and SW100-CL100 materials. This resulted in an average variation across the concrete crushing safety factor of less than 10%. The similarities between the linear elastic model and Duncan model could be due to the low cover depths assessed (please note that low cover depths in this instance refers to depths less than approximately 1.8 m rather than the low cover depths of 300 mm assessed in the design scenarios) (CNAConsultingEngineers et al., 2009). CNAConsultingEngineers et al. (2009) determined that bending moments and thrusts in rigid culverts were similar between linear elastic, Mohr Coulomb and a hardening soil model for low cover depths. However, the difference between the safety factor for shear failure between the Mohr Coulomb and Duncan model were due to the altered calculation for maximum shear strength. The linear elastic and Duncan models both had constant shear strength across all pipe nodes, whereas the Mohr Coulomb model utilised the Heger Mcgrath method to determine the shear strength based upon the applied moment, thrust and shear at the pipes circumference (Heger & McGrath, 1982).

Potential drawbacks of the more complex Duncan model arise due to the required input parameters associated with the material types. In order to utilise materials which do not have defined Duncan model parameters triaxial testing would need to be undertaken on the required material (Ti, Huat, Noorzaei, Jaafar, & Sew, 2009). This limits the usefulness of the model for assessing the various alternative backfills without predefined parameters, although, the library of existing material parameters for granular materials is considerable (Katona, 2015). The Mohr Coulomb models main drawback is the non-convergence caused by unrestrained plastic flow from the surface live load. This live load places high stresses at the surface which can lead to shear failure. This is considered to be the reason for non-convergence of the SW100 and CA105 backfills throughout all three cover depths. These materials are the only ones without any cohesion and as such would have a limited maximum shear stress (Das, 2010).

A comparison between the AS/NZS 3725 design method and the FEM in CANDE identified that there was a large difference between the safety factors against failure for the 300 mm cover depth. The AS/NZS 3725 method indicated that all scenarios will fail. However, only one scenario for the FEM models indicated potential failure. This was the SW100-SM100 backfill with the linear elastic model, which returned a value of 0.966 for shear failure, whereas the AS/NZS 3725 method returned all scenarios less than 0.65. This identified the conservatism within the standard method for low cover depths (less than 400 mm) due to the altered method of live load analysis. Similar simplified design equations (SDE) are utilised in AASHTO standards and load and resistance factor design specifications in USA where NCHRP (2010) propose improvements to the SDE via a comparison to 2D and 3D FEM modelling. The AS/NZS 3725 method also appeared to be un-conservative for the higher cover depths with HS3 type conditions for the granular materials while conservative for CLSM, stabilised sand and open grade aggregate. However, this is identified as the strength of such FEM programs by Duane et al. (1986), with their ability to model scenarios outside of those specified in standards. This is of particular relevance where the bedding factors applied within AS/NZS 3725 rely upon early work by Marston and Spangler and various researchers who focused upon various granular backfill materials (Committee:WS-006, 2007b). The overestimation of safety factors for the AS/NZS 3725 method with HS3 conforming support could also be due to the difference in material parameters used for the development of the standard and those utilised within this research.

6.2 Backfill Materials

From the FEM analysis of the various backfill materials an obvious trend was identified throughout the scenarios. This was the largely increased safety factor obtained by utilising stabilised sand backfill as well as CLSM. These materials also resulted in reduced deflections within the backfill material. Such properties are desirable for culvert backfill and correspond with findings from Kaneshiro, Navin, Wendel, and Snowden (2001) and Zhan and Rajani (1997). These materials may, however, have increased upfront cost opposed to general granular materials or aggregates. Although, construction costs may be reduced given the potential for reduced testing requirements and improved backfill speeds. The open grade aggregate materials (No6 and No8) also improved the safety factors against failure while reducing deflections, just not to the same extent as the CLSM and stabilised sand. Similarly, Jayawickrama, Amarasiri, Region, and Alam (2001) found that coarse gravels provided superior support to flexible pipe culverts while minimising the impact of poor compaction. However, consideration must be given to construction methods and placement to ensure the aggregate backfill materials performance in practice. The material must be wrapped in a geotextile to limit the migration of fines surrounding the trench and prevent future erosion around the pipe (RTA, 2009). Special attention must also be paid to the compaction methods with controlled lifts and compaction techniques specified to ensure the material does not undergo subsidence

(RTA, 2009; Tysl & Noll, 2011). It is important to note that the AS/NZS 3725 analysis method resulted in low factors of safety for the stabilised sand, CLSM and aggregate backfill in comparison to conforming granular backfill.

The alternative CLSM backfill material was also analysed for reduced trench widths due to the potential benefits that reducing trench size offers (NYSDOT, 2013; Yoo et al., 2005). This reduction in width, however, resulted in no consistent trends between each scenario. However, it was noticed that the particular material properties used resulted in consistent and beneficial results for the CLSM alternative material. This material resulted in reduced deflections and improved safety factors for reduced trench widths. From the results it was identified that a reduced trench width will not always result in improved performance and is also dependent upon specific material properties, cover depths and loading conditions.

Amongst the granular backfill materials, the conforming gravels performed the best in terms of having higher safety factors and reducing displacements. However, these materials resulted in insignificantly different shear stresses and would therefore be at the same risk of shear failure as the non-conforming granular materials. Although, the material may have a lower maximum shear strength than a clay type material with higher cohesion (Katona, 2015). This result was matched across the soil models, AS/NZS 3725 and the different soil depths. However, for the non-conforming gravels (CL95, CL100 and CA105) the CA105 retained a greater safety factor against failure and lower displacements for each of the soil models except the Mohr Coulomb model. This was due to the reduced non-convergence of the Mohr Coulomb model for the CL materials and reduced likelihood of unrestrained flow due to the high cohesion values (where CA105 had zero cohesion) and higher maximum shear stress (Das, 2010). The CA105 material also returned a slightly lower factor of safety for the AS/NZS 3725 analysis method. This was due to the higher unit weight of the material, as all other steps in the Australian Standard analysis method are the same between non-conforming granular materials.

As mentioned, the nonconforming gravels with high plasticity tended to have higher deflections and lower safety factors. These materials are also potentially problematic due to their potential to retain water and shrink/swell (Nataatmadja & Kumar, 2009). This can result in deformations around the pipe and lead to increased water ingress into the pavement material due to increased retention times within the pipe backfill. This material may also require increased effort to properly compact the material (Dasgupta, 2014; Melbourne Rail Water Agencies, 2013). Hence, the requirement for backfill material to be free draining and not to break down when exposed to wetting and drying by Committee:WS-006 (2007a).

Two problems encountered during the linear elastic analysis were from the 1200 mm cover CLSM 28day and the stabilised sand, which did not follow the trends of the previous layers. The CLSM had a slightly higher X displacement than the 600 mm layer, however, the increase was only minor and does not indicate that the model is functioning incorrectly. The results for the stabilised sand showed a lower safety factor than the 300 and 600 mm cover depths. The result was exactly one and indicated a failure of the material at this depth. However, this was not consistent with the remaining results. The Mohr Coulomb model also had inconsistent results for the SW100-SW100 and stabilised sand which were due to non-convergence of the models, as discussed in section 6.1. This inconsistency related to the improper calculation of deflections for the SW100-SW100 material which returned negative (upward) surface deflections. The stabilised sand material's 300 mm cover depth scenario did not follow the trends of the other material cover depths for all deflections, moments and safety factors. The Duncan model results behaved in a consistent and expected matter in most cases.

6.3 Case Studies

The results from case study 1 referring to the 750 mm culvert backfilled with aggregate indicated a beneficial reduction to the structural loading on the pipe. This is consistent with the results of the scenario comparison. Once again the drawback of the Duncan model not being able to assess alternative backfill materials was highlighted within the case studies. The AS/NZS 3725 results indicated that the alternative backfill method was acceptable, but with a lower factor of safety compared to utilising non-conforming gravel. This followed the trends identified within the overall comparison. Also the linear elastic and Mohr Coulomb models indicated the potential for increased displacements due to the utilisation of the aggregate backfill, reflecting the concerns raised by RTA (2009). The results are backed up by the culverts performance in the field with no cracking or physical deformation being observed. Data is not available on whether excess deflections or movement around the culvert has occurred.

Case study 2 differed from case study 1 as the AS/NZS 3725 analysis indicated failure of the pipe for all construction cases. However, the models all indicated acceptable performance of conforming granular backfill, except for the Duncan Model. In this case the AS/NZS 3725 and Duncan results correctly identified the potential for failure of the pipe with the original conforming backfill failing. This confirms the ability of the Duncan model to more realistically replicate real world results. Although, it is important to note that the original pipes were older and their existing structural integrity was unknown. However, following replacement and backfilling with stabilised sand no further failures were observed. To date the culvert has performed satisfactorily showing no signs of failure, erosion or movement. This performance matches the results indicated by the FEM models for the alternative backfill, but disagrees with the AS/NZS 3725 design method. Once again the Mohr Coulomb model encountered convergence as discussed previously. This non convergence resulted in excessively high deflections however, the remaining results appear relatively consistent.

The effect of construction techniques upon culvert loading were only assessed for these two specific cases. Further research was planned in order to assess the effects overall on the RMS northern region network however, this was determined to be unfeasible. This is because the link between culvert construction methods and potential failure is often difficult to gauge due to the impact of environmental conditions and external effects. In order to assess these external effects and gauge the impacts of construction techniques long term study and monitoring of the culvert performance would be required. This would enable a link to be drawn between the defect or failure of the pipe and the cause. Current network data does not provide this level of assessment. Such assessment would also enable information to be collected upon the materials effects of long term erosion, and the impacts of water on the surrounding pavement.

7 Conclusion

7.1 Project Conclusions

Analysis of the geotechnical aspects of pipe culvert construction can promote an understanding of the impact that varying construction practices can have on culvert structures. Various bedding and backfill materials are available for different purposes with CLSM, stabilised sand, aggregate, non-conforming granular material and conforming granular material all being suitable for culvert construction. The different methods of placing and compacting these materials was outlined with granular materials and stabilised sand requiring conventional compaction processes, aggregate requiring confinement and alternative compaction and CLSM being self-compacting.

Based upon several design scenarios it was identified that for a range of cover from 0.3 to 1.2 m and a W80 wheel load the highest cover provided the least transfer of load to the pipe and is generally favourable. For most cases, the bedding and backfill material that best limited the structural impact on the pipe was the stabilised sand. However, CLSM provided improved results over the granular materials as well. Research also identified the trench condition as being generally superior, in reducing pipe loading, to an embankment construction condition due to the interaction of the backfill and trench sidewalls.

Research and a comparison of design methodologies identified significant drawbacks of the AS/NZS 3725 design method when assessing non-conforming materials in comparison to modelled results. The Australian Standard method provided a reduced safety factor for low cover depths, compared to the FEM models. This was due to the load being applied directly on the pipe at cover depths below 300 mm. However, an over estimation of live loading at higher cover depths also generally reduces the potential suitability of certain design scenarios. This was evident for the conforming granular materials identified to provide HS3 support. The Duncan model has been identified by the literature to be the superior soil model, which was confirmed by the results of case study 2. However, certain limitations do exist given the limited ability to assess alternative materials without the required material parameters for the model being readily available. Utilising a Mohr Coulomb model also provided problems with non-convergence due to the inability to model the large surface live load.

Through an analysis of two alternative construction techniques within the Northern region of the RMS it was determined that the alternative backfill was generally favourable to the conforming material. However, the AS/NZS 3725 design method did not reflect this conclusion.

It was also found that the performance of the aggregate backfill was highly dependent upon construction methods and has increased potential for movement and erosion.

7.2 Future Research

When assessing the effect of geotechnical characteristics of culvert construction on the design process there are several paths that require further research. The scenarios not covered within this research could be compared in order to assess the effects of deep cover depths, different culvert sizes and other installation conditions. Similar studies could also be undertaken for assessing the response of flexible culverts to these alternative construction methods. This would involve analysis by the Australian Standard, FEM programs and in field testing to assess the response to loading. In FEM analysis further development is needed in order to develop material parameters for alternative backfill that can be utilised with the Duncan model. This would involve the physical testing and development of material parameters for the alternative backfill. Verification of the ability of FEM programs to assess such alternative backfill materials is also required with comparisons to alternative programs and physical testing allowing for more accurate results. Further development into assessing the impacts of alternative construction techniques, not covered by the standard, is also required. This includes the impact of sloping or benched trench walls on culvert loading.

Research relating to the evaluation of the identified construction techniques, assessing their advantages and disadvantages as well as the overall cost of the various backfilling methods would be of benefit. This assessment could consider the potential time and cost savings that alternative materials can allow for as well as the environmental costs associated with the various materials. Further study on these techniques is required to understand the performance in the field, identifying the potential for water ingress into the pavement, erosion and other failure mechanisms. This research could then be incorporated into current practices, specifications and standards in order to enable less restrictive practices and benefit from improved construction techniques.

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Appendix A: Project Specification

UNIVERSITY OF SOUTHERN QUEENSLAND

FACULTY OF ENGINEERING AND SURVEYING

ENG4111/4112 RESEARCH PROJECT
PROJECT SPECIFICATION

FOR: **Simon James PORTER**

TOPIC: Analysing Geotechnical Aspects of Concrete Pipe Culverts

SUPERVISOR: Kazem Ghabraie

SPONSORSHIP: Roads and Maritime Services NSW

PROJECT AIM: Compare and assess the effects of construction and design methods for concrete pipe culverts.

PROGRAMME: **Issue A, 08 March 2015**

1) Research the various materials and methods utilised for placing the bedding and backfilling pipe culverts.
2) Research the various analysis techniques for assessing the load distribution upon pipe culverts in various conditions.
3) Determine several scenarios to compare individual culvert construction and load assessment measures.
4) Identify best case scenarios and limitations of the various methods as well as assessing the current safety factors in design.

As time permits:

5) Collect and compile current culvert inventory data on RCPC within the Northern NSW RMS region to assess the causes of existing failures.
6) Analyse the contribution that construction methods and geotechnical conditions contributed to the failures identified.

AGREED ______________________ (Student) ______________________ (Supervisor)

Date: Date:

Examiner/Co-examiner: ______________________________

Appendix B: AS/NZS 3725 Calculations

Table B 1: AS/NZS 3725 Calculations 300 mm Depth

AS/NZS 3725 Design Procedure (300 mm depth)								
Material	Trench Width B (m)	Ct (Cl 6.3.2)	Wb (kN/m)	wq (kN/m)	F	Fq	Tc (kN/m)	SF (CIV)
CLSM (1day)	0.8	0.35	4.43	188.24	2	1.5	127.70	0.61
	0.9	0.31	5.02	188.24	2	1.5	128.00	0.61
CLSM (7day)	0.8	0.35	4.17	188.24	2	1.5	127.57	0.61
	0.9	0.31	4.72	188.24	2	1.5	127.85	0.61
CLSM (28day)	0.8	0.35	3.68	188.24	2	1.5	127.33	0.61
	0.9	0.31	4.17	188.24	2	1.5	127.58	0.61
CLSM (alternative)	0.8	0.35	4.88	188.24	2	1.5	127.93	0.61
	0.9	0.31	5.53	188.24	2	1.5	128.25	0.61
No6	0.9	0.31	4.40	188.24	1.275	1.275	151.09	0.52
No8	0.9	0.31	4.50	188.24	1.275	1.275	151.17	0.52
SM100-CA105	0.9	0.31	4.98	188.24	1.275	1.275	151.54	0.51
SM100-CL95	0.9	0.32	4.62	188.24	1.275	1.275	151.26	0.52
SM100-CL100	0.9	0.32	4.70	188.24	1.275	1.275	151.33	0.52
SM100-SM100	0.9	0.31	4.78	188.24	4	1.5	126.69	0.62
SM100-SW100	0.9	0.31	4.98	188.24	4	1.5	126.74	0.62
Stab Sand	0.9	0.31	5.98	188.24	2	1.5	128.48	0.61

Table B 2: AS/NZS 3725 Calculations 600 mm Depth

AS/NZS 3725 Design Procedure (600 mm depth)								
Material	Trench Width B (m)	Ct (Cl 6.3.2)	Wb (kN/m)	Wq (kN/m)	F	Fq	Tc (kN/m)	SF (CIV)
CLSM (1day)	0.8	0.65	8.27	31.21	2	1.5	24.94	3.13
	0.9	0.59	9.44	31.21	2	1.5	25.53	3.06
CLSM (7day)	0.8	0.65	7.78	31.21	2	1.5	24.70	3.16
	0.9	0.59	8.89	31.21	2	1.5	25.25	3.09
CLSM (28day)	0.8	0.65	6.87	31.21	2	1.5	24.24	3.22
	0.9	0.59	7.85	31.21	2	1.5	24.73	3.15
CLSM (alternative)	0.8	0.65	9.10	31.21	2	1.5	25.36	3.08
	0.9	0.59	10.40	31.21	2	1.5	26.01	3.00
No6	0.9	0.59	8.28	31.21	1.275	1.275	30.97	2.52
No8	0.9	0.59	8.47	31.21	1.275	1.275	31.12	2.51
SM100-CA105	0.9	0.59	9.37	31.21	1.275	1.275	31.83	2.45
SM100-CL95	0.9	0.62	8.92	31.21	1.275	1.275	31.47	2.48
SM100-CL100	0.9	0.62	9.08	31.21	1.275	1.275	31.60	2.47
SM100-SM100	0.9	0.59	9.00	31.21	4	1.5	23.06	3.38
SM100-SW100	0.9	0.59	9.37	31.21	4	1.5	23.15	3.37
Stab Sand	0.9	0.59	11.24	31.21	2	1.5	26.43	2.95

Table B 3: AS/NZS 3725 Calculations 1200 mm Depth

AS/NZS 3725 Design Procedure (600 mm depth)								
Material	Trench Width B (m)	Ct (Cl 6.3.2)	Wb (kN/m)	Wq (kN/m)	F	Fq	Tc (kN/m)	SF (CIV)
CLSM (1day)	0.8	1.14	14.48	12.09	2	1.5	15.30	5.10
	0.9	1.05	16.77	12.09	2	1.5	16.45	4.74
CLSM (7day)	0.8	1.14	13.63	12.09	2	1.5	14.88	5.24
	0.9	1.05	15.79	12.09	2	1.5	15.95	4.89
CLSM (28day)	0.8	1.14	12.04	12.09	2	1.5	14.08	5.54
	0.9	1.05	13.95	12.09	2	1.5	15.04	5.19
CLSM (alternative)	0.8	1.14	15.95	12.09	2	1.5	16.04	4.86
	0.9	1.05	18.47	12.09	2	1.5	17.30	4.51
No6	0.9	1.05	14.71	12.09	1.275	1.275	21.02	3.71
No8	0.9	1.05	15.05	12.09	1.275	1.275	21.29	3.66
SM100-CA105	0.9	1.05	16.64	12.09	1.275	1.275	22.54	3.46
SM100-CL95	0.9	1.16	16.62	12.09	1.275	1.275	22.52	3.46
SM100-CL100	0.9	1.16	16.91	12.09	1.275	1.275	22.75	3.43
SM100-SM100	0.9	1.05	15.98	12.09	4	1.5	12.06	6.47
SM100-SW100	0.9	1.05	16.64	12.09	4	1.5	12.22	6.38
Stab Sand	0.9	1.05	19.97	12.09	2	1.5	18.05	4.32

Table B 4: AS/NZS 3725 Case Study Calculations

Study	**Trench Width B (m)**	**Ct (Cl 6.3.2)**	**Wb (kN/m)**	**wq (kN/m)**	**F**	**Fq**	**Tc (kN/m)**	**SF (CIV)**
Case 1 (Conforming)	0.9	0.41	6.25	56.36	4	1.5	39.14	2.45
Case 1 (Aggregate)	0.9	0.41	5.75	56.36	1.275	1.275	48.72	1.97
Case 2 (Conforming)	0.9	0.31	4.78	377.22	4	1.5	252.68	0.31
Case 2 (Stabilised sand)	0.9	0.31	5.98	377.22	2	1.5	254.47	0.31

Appendix C: Node 56 Comparison Data

Table C 1: 300 LE Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
CLSM (1day)	-2.48	-18.06	2.41	3.89	0.83
CLSM (7day)	-2.38	-17.93	3.52	20.35	0.77
CLSM (28day)	-1.89	-16.41	11.65	8.26	0.34
CLSM (alter)	-2.25	-17.79	6.71	16.82	0.63
No6	-3.81	-18.13	-3.89	41.61	1.60
No8	-3.83	-18.13	-3.97	41.42	1.58
SM100-CA105	-4.67	-17.93	-4.51	45.43	1.78
SM100-CL95	-5.32	-18.34	-4.17	49.28	2.03
SM100-CL100	-5.13	-18.27	-4.18	48.60	1.98
SM100-SM100	-4.99	-18.20	-4.29	47.59	1.91
SM100-SW100	-4.73	-17.93	-4.54	45.81	1.81
SW100-SW100	-4.60	-18.00	-4.66	45.53	1.80
SW100-CA105	-4.51	-18.00	-4.56	45.40	1.79
SW100-CL95	-5.10	-18.48	-4.29	49.23	2.03
SW100-CL100	-4.95	-18.41	-4.34	48.45	1.97
SW100-SM100	-3.65	-18.62	-0.67	55.79	2.33
Stab Sand	-1.80	-16.13	13.69	7.18	0.33

Table C 2: 300 DS Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
SM100-CA105	-4.62	-18.96	-6.52	46.23	1.88
SM100-CL95	-5.30	-20.62	-2.63	50.04	2.13
SM100-CL100	-5.21	-21.65	-6.00	47.71	2.02
SM100-SM100	-4.58	-19.93	-6.57	44.81	1.84
SM100-SW100	-4.38	-17.93	-5.47	45.05	1.75
SW100-SW100	-4.59	-18.13	-5.89	44.86	1.73
SW100-CA105	-4.85	-19.17	-6.75	46.10	1.86
SW100-CL95	-5.60	-20.75	-2.82	49.87	2.09
SW100-CL100	-5.49	-21.79	-6.12	47.66	2.01
SW100-SM100	-4.85	-20.13	-6.63	44.87	1.85

Table C 3: 300 MC Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
CLSM (alter)	-2.09	1.37	2.16	37.53	0.07
No6	-3.92	-18.96	-6.73	37.50	1.65
No8	-3.79	-18.48	-5.39	38.61	1.61
SM100-CA105	-5.58	-21.24	-10.90	43.12	1.75
SM100-CL95	-5.82	-20.55	-6.14	44.61	1.96
SM100-CL100	-5.58	-19.99	-5.90	43.58	1.91
SM100-SM100	-5.31	-18.82	-5.19	44.53	1.86
SM100-SW100	-5.75	-20.89	-11.21	44.09	1.80
SW100-SW100	-5.49	-20.89	-10.31	46.02	1.89
SW100-CA105	-5.48	-20.89	-11.05	47.58	1.92
SW100-CL95	-5.48	-20.34	-5.69	44.73	1.99
SW100-CL100	-5.27	-19.86	-5.48	43.86	1.95
SW100-SM100	-5.03	-18.96	-4.79	44.36	1.90
Stab Sand	-2.59	-18.89	-0.21	11.04	1.19

Table C 4: 600 LE Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
CLSM (1day)	-1.65	-18.20	0.63	1.21	0.37
CLSM (7day)	-1.54	-17.58	0.75	8.55	0.46
CLSM (28day)	-1.35	-15.93	3.67	2.36	0.14
CLSM (alter)	-1.56	-17.58	2.10	4.95	0.28
No6	-2.23	-21.79	-1.74	15.56	0.75
No8	-2.25	-21.86	-1.81	15.40	0.74
SM100-CA105	-2.68	-22.82	-2.15	17.28	0.83
SM100-CL95	-2.93	-22.89	-2.34	18.71	0.94
SM100-CL100	-2.87	-22.96	-2.31	18.41	0.91
SM100-SM100	-2.81	-23.03	-2.26	18.13	0.89
SM100-SW100	-2.71	-22.89	-2.19	17.44	0.84
SW100-SW100	-2.63	-22.68	-2.25	17.47	0.85
SW100-CA105	-2.61	-22.68	-2.20	17.30	0.84
SW100-CL95	-2.82	-22.75	-2.43	18.74	0.94
SW100-CL100	-2.77	-22.75	-2.37	18.46	0.93
SW100-SM100	-2.71	-22.82	-2.34	18.16	0.89
Stab Sand	-1.38	-16.13	4.60	1.44	0.10

Table C 5: 600 DS Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
SM100-CA105	-2.43	-22.89	-2.30	17.15	0.79
SM100-CL95	-2.75	-22.48	-2.39	18.83	0.93
SM100-CL100	-2.84	-24.55	-3.82	18.60	0.97
SM100-SM100	-2.41	-23.99	-2.59	16.93	0.83
SM100-SW100	-2.43	-22.55	-2.22	16.65	0.77
SW100-SW100	-2.59	-22.34	-2.35	16.57	0.75
SW100-CA105	-2.61	-22.68	-2.39	17.06	0.79
SW100-CL95	-2.99	-22.20	-2.25	18.87	0.95
SW100-CL100	-3.09	-24.27	-3.80	18.58	0.97
SW100-SM100	-2.60	-23.72	-2.77	16.79	0.81

Table C 6: 600 MC Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
CLSM (alter)	-1.46	0.85	2.16	11.75	0.02
No6	-2.22	-22.27	-2.22	13.69	0.74
No8	-2.14	-23.24	-0.08	15.86	0.78
SM100-CA105	-3.34	-25.58	-5.99	16.27	0.86
SM100-CL95	-3.13	-23.65	-2.15	18.62	0.93
SM100-CL100	-3.04	-23.37	-2.13	18.25	0.90
SM100-SM100	-2.97	-23.30	-2.26	17.89	0.87
SM100-SW100	-3.37	-25.51	-5.94	16.75	0.88
SW100-SW100	-3.16	-25.30	-5.70	16.79	0.90
SW100-CA105	-3.13	-25.37	-5.74	16.63	0.89
SW100-CL95	-2.97	-23.51	-2.17	18.62	0.94
SW100-CL100	-2.90	-23.30	-2.18	18.26	0.90
SW100-SM100	-2.84	-23.10	-2.29	17.95	0.88
Stab Sand	-2.73	-30.89	-4.95	27.04	1.37

Table C 7: 1200 LE Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
CLSM (1day)	-1.20	-17.03	-0.87	0.80	0.29
CLSM (7day)	-1.16	-16.69	-0.69	4.08	0.26
CLSM (28day)	-1.05	-15.44	0.65	1.56	0.13
CLSM (alter)	-1.22	-16.96	-0.05	2.34	0.17
No6	-1.44	-19.37	-1.29	7.25	0.38
No8	-1.46	-19.51	-1.34	7.20	0.38
SM100-CA105	-1.71	-20.48	-1.41	8.19	0.43
SM100-CL95	-1.77	-20.48	-1.36	8.61	0.46
SM100-CL100	-1.75	-20.48	-1.37	8.55	0.45
SM100-SM100	-1.75	-20.62	-1.39	8.52	0.45
SM100-SW100	-1.72	-20.48	-1.42	8.26	0.43
SW100-SW100	-1.67	-20.41	-1.45	8.30	0.44
SW100-CA105	-1.66	-20.41	-1.44	8.22	0.43
SW100-CL95	-1.71	-20.41	-1.40	8.64	0.46
SW100-CL100	-1.70	-20.41	-1.41	8.58	0.45
SW100-SM100	-1.69	-20.48	-1.43	8.55	0.45
Stab Sand	-1.16	-16.06	1.46	0.87	0.08

Table C 8: 1200 DS Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
SM100-CA105	-1.49	-19.79	-1.69	7.31	0.38
SM100-CL95	-1.55	-19.93	-1.35	8.29	0.44
SM100-CL100	-1.67	-21.17	-2.18	9.13	0.49
SM100-SM100	-1.45	-20.20	-1.39	7.32	0.38
SM100-SW100	-1.52	-19.99	-1.63	7.68	0.40
SW100-SW100	-1.68	-19.79	-1.65	7.59	0.39
SW100-CA105	-1.64	-19.58	-1.72	7.23	0.38
SW100-CL95	-1.74	-19.65	-1.30	8.26	0.44
SW100-CL100	-1.87	-20.96	-2.12	9.10	0.49
SW100-SM100	-1.60	-19.99	-1.38	7.28	0.38

Table C 9: 1200 MC Node 56

Backfill Type	Max Deflection (Y) (mm)	Max Soil Stress (kPa)	Max Thrust (kN/m)	Max Shear (kN/m)	Max Moment (kN/m)
CLSM (alter)	-1.15	0.27	2.16	-0.46	0.01
No6	-2.03	-17.51	-2.00	8.92	0.49
No8	-1.46	-19.37	-1.34	7.15	0.38
SM100-CA105	-2.13	-21.86	-3.76	8.91	0.48
SM100-CL95	-1.89	-20.34	-1.41	8.51	0.45
SM100-CL100	-1.87	-20.27	-1.42	8.44	0.44
SM100-SM100	-1.86	-20.48	-1.45	8.42	0.44
SM100-SW100	-2.15	-21.86	-3.71	9.05	0.49
SW100-SW100	-2.03	-21.51	-3.58	9.11	0.50
SW100-CA105	-2.02	-21.58	-3.61	8.94	0.50
SW100-CL95	-1.80	-20.34	-1.44	8.55	0.45
SW100-CL100	-1.79	-20.34	-1.45	8.49	0.45
SW100-SM100	-1.78	-20.41	-1.48	8.46	0.44
Stab Sand	-1.61	-23.65	-3.38	10.04	0.69