University of Southern Queensland

Faculty of Health, Engineering and Sciences

Flexural Investigation and Design of Stressed-Skin Panels With an Autoclaved Aerated Concrete (AAC) Core

A dissertation submitted by

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In fulfilment of the requirements of

ENG4111 and 4112 Research Project

towards the degree of

Bachelor of Engineering (Honours) (Civil)

To Be Submitted October, 2019

Abstract

This dissertation has been created with the desire to undertake an investigation and subsequent design of stressed-skin panels with an autoclaved aerated concrete (AAC) core which considers different material systems for the stressed-skin component in order to determine its potential applications in the Australian building industry. The motive for the project stems from clear industry concern for the safety and certification of composite structures (such as stressed-skinned panels) in relation to the fire performance of some core materials such as the widespread use of typical foams. The issues with existing systems and the potential benefits of using AAC were supported by the literature review which reinforces the need and importance for the industry and it's supporting engineers to consider innovative ideas that can help to improve the safety and reliability of buildings.

Four-point load testing was completed on four test samples with variation in core thickness (75mm and 150mm), skin type (no skin, GFRP and steel) and span distance (2000mm and 5340mm) used to understand the behaviour of the structure once loaded. The testing proved successful as the data showed that the stressed-skin panels supported the loading conditions well over different spans and displayed a significantly improved performance over a skinless AAC panel of the same configuration.

The potential to use the system as a wall, roof, floor or permanent formwork was highlighted and the relevant Australian standards and codes were used to develop application specific criteria for serviceability and ultimate strength conditions. Analytical methods were adopted and calibrated against the testing data in order to predict the load carrying capabilities of each application with variation in span (1.5m to 6.0m), core density (400kg/m^3 – 580kg/m^3), core thickness (75mm to 200mm) and steel skin thickness (0.4mm to 2.0mm) explored. The results were presented in graphical form with corresponding span tables and idealised arrangements were determined for each application with a measure of efficiency between core density and skin thickness. A summary of the most efficient (and therefore recommended) combinations can be given as:

- **Wall** → 510kg/m^3 core with 0.40mm skins and 550kg/m^3 core with 0.55mm skins
- **Roof** → 400kg/m^3 core with 0.40mm skins, 510kg/m^3 core with 0.55mm skins and 550kg/m^3 core with 0.95mm skins
- **Floor** → 510kg/m^3 core with 0.55mm skins and 550kg/m^3 core with 0.95mm skins
- **Formwork** → 550kg/m^3 core with 0.40mm skins

It was found the concept of a steel stressed-skin panel with an AAC core generally satisfies the performance criteria for the building applications considered, and if accompanied with further work, has the potential to make a positive change in the Australian building industry.

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Limitations of Use

Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Acknowledgements

I would like thank everyone that has been involved in any way throughout the process of developing and completing this dissertation, in particular my supervisor My Gary Elks for providing the inspiration for this topic and his commitment throughout the year to help guide me through the project. I would also like to thank Associate Professor Allan Manalo for this technical assistance throughout the analysis. Finally, I would like to thank my family for their continual and exceptional support throughout this year and for all of the preceding study leading up to this point. Without them I would not be in a position to reach this achievement.

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Nomenclature

A	Distance from Support to Applied Load (mm)
b	Width of Panel (mm)
C	Material Heat Capacity (J/K)
c	Core Thickness (mm)
c_1	Distance of Extreme Fibers from Centroid (mm)
c_2	Half of Core Thickness (mm)
d	Depth of Structure (mm)
E	Modulus of Elasticity (MPa)
E_c	Modulus of Elasticity of Core Material (MPa)
E_d	Design Action Effect
E_u	Ultimate Earthquake Action
E_s	Modulus of Elasticity of Skin Material (MPa)
E_{serv}	Serviceability Earthquake Loading
EI	Flexural Rigidity (kN/m^2)
G	Dead Load (kPa)
G_c	Concrete Load (kPa)
I	Second Moment of Area (m^4)
k	Material Conductivity (W/mK)
L	Span (mm)
M	Bending Moment Force (kN.m)
M_1	Load From Stacked Materials During Stage 1 of The Construction Cycle
M_2	Load From Stacked Materials During Stage 2 of The Construction Cycle
M_3	Load From Stacked Materials During Stage 3 of The Construction Cycle
P	Applied Load (kN)

P_f	Failure Load (kN)
Q	Imposed Load (kPa)
Q_A	Statical Moment of Area (m^3)
Q_{uv}	Uniformly Distributed Vertical Live Load (kPa)
R	Reaction at Support (kN)
R_d	Design Capacity
S_u	Ultimate Value Of Various Actions Appropriate For Particular Combinations
t_c	Thickness of the Core Material (mm)
t_s	Thickness of the Skin Material (mm)
V	Shear Force (kN)
V_m	Shear Modulus (MPa)
w	Uniformly Distributed Load (kN/m)
W_s	Serviceability Wind Action
W_u	Ultimate Wind Action
σ_c	Bending Stress Capacity of Core Material (MPa)
σ_s	Bending Stress Capacity of Skin Material (MPa)
$\sigma_{s(c)}$	Bending Stress Capacity of Skin Material in Compression (MPa)
$\sigma_{s(t)}$	Bending Stress Capacity of Skin Material in Tension (MPa)
ψ_c	Combined Factor
ψ_E	Combination Factor for Earthquake Actions
ψ_l	Long-Term Factor
ψ_s	Short-Term Factor
Δ	Deflection (mm)
τ_c	Shear Capacity of Core (kN)
δ_l	Limiting Value of The Serviceability Parameter

ρ	Material Density (kg/m^3)
δ	Value of The Serviceability Parameter Determined On The Basis Of Design Actions
$\emptyset$	Reduction Factor

Glossary of Terms

AAC	Autoclaved Aerated Concrete
ISP	Insulated Sandwich Panels
EPS	Expanded Polystyrene
PUR	Polyurethane Foam
PIR	Polyisocyanurate
EPS-FR	Expanded Polystyrene Fire Resistant
MRF	Mineral Fibre
XPS	Extruded Polystyrene
SPS EPS	Phenolic Hybrid – Syntactic
NCC	National Construction Code
FRL	Fire Resistance Level
FRAC	Fibre-Reinforced aerated concrete
ARG	Alkali Resistant Glass
CFD	Computational Fluid Dynamic
GFRP	Glass Fibre Reinforced Polymer

1 Introduction

1.1 Background

The concept of a stressed-skin panel, also known as a sandwich panel, has been established for a significant period of time and over the course of history it has been implemented in areas such as aircraft, automotive vehicles, trains and sports equipment and is even evident in nature through examples such as the human skull, bird wings and the stalk and leaves of some plants (Ashby 2011). A stressed-skin panel system generally consists of a lightweight core material which is sandwiched between two thin skin materials, which combine to create a relatively lightweight and efficient structure. With the many inherit and desirable performance characteristics that sandwich panels contain, it presents itself as a useful product in many building applications such as structural and architectural floors, roofs and wall systems.

Some core materials currently used in this style of construction, such as foam, possess a potential fire hazard with poor flammability and toxicity properties of the core materials being used. This has created concern in the building industry and hence generated a degree of hesitancy around using such products. An example of such a hazard occurred in 2010 at a food processing factory in Melbourne, where a fire developed and quickly spread to the expanded polystyrene (EPS) sandwich ceiling panel and was able to spread through to the roof over half of the length of the building. The findings from this particular example established that the fire was able to spread inside the ceiling panels prior to bursting out at the panels joints which progressively delaminated whilst fuelling the fire. The other issues that were established from the investigation included the risk of major steel roof beams collapsing as a direct result of the fire load from the sandwich panels and the enquiry also highlighted the risk that fire fighters face upon entering a building with the potential of collapsing (Zurich 2015).

Similar examples across the globe such as the Grenfell Tower fire in West London has resulted in the fire safety of cladding and sandwich systems becoming a topical and controversial subject. The issues have flowed into the Australian construction industry with builders, certifiers, developers and even the general public having major concerns about safety and certification of new and existing structures to the point where there is a negative perception and stigma around the use of composite panel systems. Whilst technology in this area has improved with modern practice in relation to fire protection, it also presents an area in which engineers have the responsibility and need to investigate alternative materials that have the ability to increase the safety of new buildings for all parties, including the occupants, insurers, emergency assistance and even the general public. This dissertation is aimed at investigating a different approach in relation to the core materials of such systems to make advancements toward creating a safe product that has the confidence of consumers and all of the associated parties.

Autoclaved aerated concrete is a cellular structure lightweight concrete material with characteristics and properties that are well suited to the principles and preferences of sandwich panel core materials whilst addressing the potential risks with existing core materials. This study is a first step to investigate and design stressed-skin panels with an autoclaved aerated concrete (AAC) core, and with different material systems for the stressed-skin to determine its potential applications in the Australian building industry.

1.2 Aim and Objectives

This dissertation sets out to address a number of different aims and objectives with some standalone aspects and others that flow on from the preceding findings. Refer to Appendix A for the detailed project specifications.

Firstly, the initial chapters will set out to review the existing stressed-skin panels and their applications that currently exist in the building industry and investigate the safety and certification issues with the commonly used stressed-skin panel systems.

Following on from the initial research, a review of autoclaved aerated concrete (AAC) will be undertaken in order to discover its relevant characteristics and the findings of which will be used during the analysis phase of this report. The outcomes from the literature will allow an educated evaluation of how an autoclaved aerated concrete (AAC) may be used as a core material.

With the discoveries from the aforementioned research targets, proposals for the potential applications of stressed-skin panels with an autoclaved aerated concrete (AAC) core will be made in an attempt to highlight potential areas of use in the Australian building industry. A review of the Australian Standards and other relevant literature will aim to outline the loading conditions that the proposed system will encounter, in order to establish the performance criteria. Knowing the relevant conditions for the proposed applications, design of the stressed-skin panels with an autoclaved aerated concrete (AAC) core with differing skin and core configurations will be undertaken. Considering all of the design data for each individual application, it is proposed that 'idealised' specifications will be found, giving the best core configuration and skin material for each application in the form of a table with various spans.

If time and resources permit, a potential cost analysis of the system in comparison to the existing methods of construction in the identified applications has been identified as a beneficial objective if time permits.

1.3 Overview of Dissertation

This dissertation is made up of nine main chapters, each focussing on a particular topic to work towards the outcomes and objectives. Chapter 1 gives an introduction into the topic and provides the background information and defines the motivation to pursue the topic of stressed-skin autoclaved aerated (AAC) core panels. Chapter 2 follows on from the topics presented in Chapter 1 and provides an extensive literature on the principles around stressed-skinned panels and the characteristics of AAC as a core material. The learnings from this chapter help shape the subsequent analysis and work completed in the chapters to follow.

Chapter 3 defines the methodology for the project which outlines the process which will be followed in the middle chapters of this dissertation. Chapter 4 describes the physical testing that took place, each sample's performance, failures and the outcomes that came from the tests. In Chapter 5, methods for calculating performance and failures for the testing were developed and implemented to compare against the actual characteristics gathered from the testing. The serviceability (deflection) and strength (failures) were the main items of assessment which were the focus of the findings from the analysis which will need to be considered in the successive chapters.

Chapter 6 sees the focus shift to more foretelling, with highlighting the potential areas where the stressed-skin AAC core panel could be utilised. In particular, this chapter provides the description of each application and the factors for each that form the criteria for design and details the variables that will be explored in the predictive analysis. Chapter 7 relates the theory established in Chapter 5 and adapts it to suit realistic loading conditions and details the decision-making processes that are used to generate the span tables.

The results and discussion are presented in Chapter 8, more specifically the span tables of interest and the prevalent discoveries from the analysis. The final section, Chapter 9, provides the conclusion to the project whilst relating to the original objectives to the findings and closes by outlining some of the potential further work that can be done in this field of work.

1.4 Scope of Works

This study will attempt to understand and apply the appropriate Australian practice, codes and standards and other requirements where appropriate for the potential applications for the proposed concept, in an attempt to establish both valid and relevant outcomes, recommendations and proposals for future work based on the information used.

2 Literature Review

2.1 Sandwich Panel

2.1.1 Description

Allen (1969) describes the most basic sandwich panel arrangement as being comprised of three layers; with two thin, stiff, strong sheets of dense material which are separated by a thicker core low-density material which is typically not as stiff or strong in comparison to the skins. The skin and core are bonded together using either a line foaming process, separate adhesives or by mechanical fasteners and this creates a single component, with a typical arrangement shown in Figure 2.1 (Davies 2008).

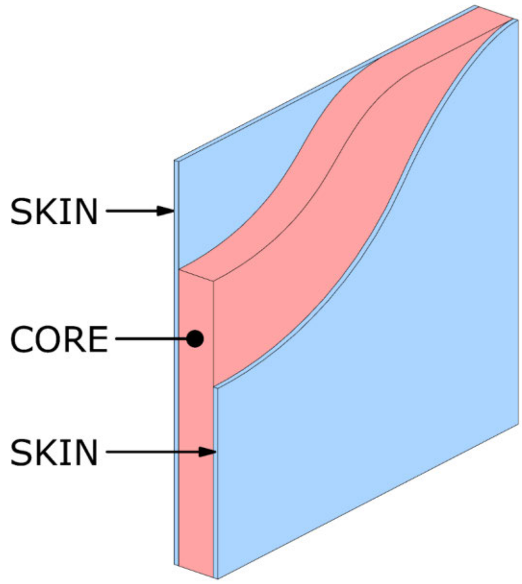

Figure 2.1 - Typical Sandwich Panel Arrangement

Australia, along with many other countries have used Insulated Sandwich Panels (ISP) for many decades in floor, wall and roofing systems (Structural Panels Australia PTY LTD 2016). Both the skin layers and core components can be made up of a number of different materials, with the most common Australian used core options being listed below (Insulated Panel Council Australasia IPCA Ltd 2019):

- EPS (Expanded Polystyrene)
- PUR (Polyurethane Foam)
- PIR (Polyisocyanurate)
- EPS-FR (Expanded Polystyrene Fire Resistant)
- MRF (Mineral Fibre)
- XPS (Extruded Polystyrene)
- SPS EPS (Phenolic Hybrid – Syntactic)

Davies (2008) outlines other potential core materials such as cork, balsa wood, rubber and paper. The key purpose of the core material is to absorb local stresses applied to one surface and reduce the effect on the structure by distributing this load and stresses over a larger area to provide an improved performance when subject to bending, torsion, impact and compression (Lamb 2010). In order to achieve its function in the sandwich panel arrangement, the core must meet three main criteria:

- In the direction perpendicular to the skins, it must be adequately stiff to ensure they are kept at the required distances apart from each other.
- Provide enough shear resistance when the panel is put into bending, to prevent the skins from slipping over each other and eliminating the effect of the sandwich arrangement.
- To ensure the skins remain sufficiently flat, the core must be stiff enough to limit the possibility of the skin materials buckling caused by compression in its own plane (Allen 1969).

The skin layers are often comprised of steel layers to benefit for their high tensile and compressive strength in conjunction with the high shear strength of the internal core (Insulated Panel Council Australasia IPCA Ltd 2019). Other skin material options may include aluminium, plywood, fibre-reinforced plastic, fibreglass, fibre cement or engineered timber (Reardon & Downton 2013).

2.1.2 Advantages

Davies (2008) describes the advantages of a particular type of sandwich panel configuration, being skins made from thin steel or aluminium skins in combination with a core of low-density plastic or mineral wool. Although there are many potential skin and core combinations, their positive characteristics remain similar and can be seen in the following list:

- High load-bearing capacity for accompanying low weight
- Good and durable thermal insulation
- Provides a water and vapour barrier
- Airtight
- Weather resistance given by skin material
- Easy installation due to lightweight nature
- Ability to repair or replace panels
- Mass production available for pre-set sizing
- Long design life
- Low maintenance cost
- Fire performance using mineral wool cores

2.1.3 Disadvantages

Davies (2008) also details the limitations of the same system, which provide elements of consideration for future work in this field, namely:

- Fire performance using plastic foam cores
- Differing exposure to one side can cause deformation of skin materials
- Creep experienced if foam cores are exposed to continual loading conditions
- Low thermal capacity
- Sound insulation due to lightweight nature
- Non-biodegradable disposal of some core materials

2.1.4 Failure Modes

Steeves and Fleck (2004) discuss the failure modes of typical sandwich panels experience when subject to three-point and four-point bending as being:

- Shearing of the core material (often the main cause of failure)
- Wrinkling of the skin materials
- Face yield (or face mircrobuckling)
- Indentation or punching of the skin caused by a concentrated loading.
- Tear-off of the skin from the core (also known as debonding) which Odessa et al. (2018) gives the causes from manufacturing deficiencies such as poor resin flow, malpractice or from low velocity impact loads under serviceability.

These failure modes have been confirmed by the research from Triantafillou and Gibson (1987), Chen et al. (2001), (Lingaiah & Suryanarayana 1991) and Odessa et al. (2018) as shown in Figure 2.2.

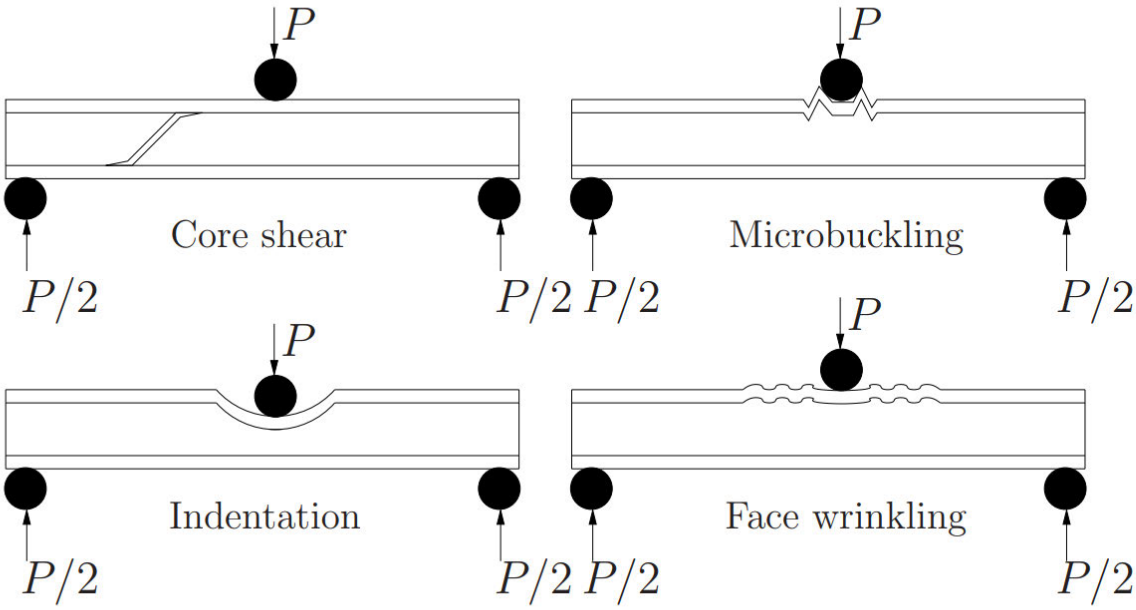

Figure 2.2 - Failure Modes

The predicted failure load for the core shear, face wrinkling, face yielding and indentation can be given by:

Core Shear Failure: $P_f = 2\tau_c bd$

Face Wrinkling Failure: $P_f = \frac{2bt_s d}{L}\sqrt[3]{E_s E_c V_m}$

Face Yielding Failure: $P_f = \frac{4\sigma_s bt_s d}{L}$

Indentaion Failure: $P_f = bt_s\left(\frac{\pi^2\sigma_c^2 E_s d}{3L}\right)^{\frac{1}{3}}$

Where:

b = Width of Panel (mm)

d = Depth of Structure (mm)

E_c = Modulus of Elasticity of Core Material (MPa)

E_s = Modulus of Elasticity of Skin Material (MPa)

L = Span (mm)

P_f = Failure Load (kN)

t_s = Thickness of the Skin Material (mm)

V_m = Shear Modulus (MPa)

σ_c = Bending Stress Capacity of Core Material (MPa)

σ_s = Bending Stress Capacity of Skin Material (MPa)

τ_c = Shear Capacity of Core (kN)

2.2 Safety Issues

2.2.1 Fire Performance

The ease in which a material can be ignited and its ability to spread a flame is known as flammability and is inversely proportional to the product of the materials conductivity (k), density (ρ) and heat capacity (C). When these properties are combined it is known as the thermal inertia and it can be stated that materials with low thermal inertia will heat more rapidly when exposed to heat, along with the characteristic of being a more rapid ignition source (Buchanan & Abu 2017). Stec and Hull (2011) provide typical technical specifications of various relevant sandwich panel core materials (described in section 2.1.1) in relation to the density, thermal conductivity range and the rating for the reaction to fire Euroclass classification as shown in Table 2.1.

Table 2.1 - Core Material Fire Properties

Material	Density Range (kg/m^3)	Thermal Conductivity Range (W/mK)	Reaction To Fire Euroclass Range
Extruded polystyrene (XPS)	20–80	0.025 – 0.035	E–F
Expanded polystyrene (EPS)	10–50	0.029–0.041	E – F
Polyurethane (PUR)	30–80	0.029–0.041	D – E
Polyisocyanurate (PIR)	30–80	0.023–0.041	C – D

Table 2.2 provides the combustibility description for each corresponding Euroclass rating, with the typical sandwich panel core materials featuring poorly towards the fire safety of a building (Apte 2006).

Table 2.2 - Euroclass Rating Descriptions

Euroclass Rating	Combustibility Description
A1 + A2	No contribution to a fire/non-combustible
B	Very limited contribution to a fire: • Heat propagation • Flame propagation • Smoke propagation
C	Limited – but some – contribution to a fire
D	Not negligible contribution to a fire
E	Poor fire reaction properties • Acceptable ignitability • Limited flame propagation
F	No performance determined – no data available

2.2.2 Toxicity

Stec (2017) explains that the toxicity of fire effluents are the largest single contributor to the number of deaths caused by unwanted fires, and the commonly used synthetic polymers burn at a fast rate and produce additional smoke and toxic effluents, particularly when exacerbated with halogenated flame retardants.

Stec and Hull (2011) highlight carbon monoxide (CO) as one of the more toxicologically substantial components in gases produced by fires as it averts oxygen transport along with hydrogen cyanide (HCN) which prevents uptake of oxygen by the cells. Hyperventilation stirred by carbon dioxide (CO_2) worsens the risk of toxic intake as a result of the increase in respiration frequency and once the body is subject to oxygen concentration levels of less than 14% fatalities will occur.

The study compared the toxicity of six insulation materials in terms of the makeup of individual toxic components (with varying ventilated conditions) using the fractional effective does (FED) model and LC_{50} (mass per unit required to create a lethal toxic atmosphere). The order of the materials in an order of increasing fire toxicity were detailed as stone wool, glass wool, polystyrene, phenolic, polyurethane and finally the polyisocyanurate foam being the most toxic material tested.

2.2.3 Australian Standards & Certification

Heath and Nguyen (2018) describes the possibilities and limitations for fire hazard reduction in regard to composite sandwich panel systems. Any sandwich panel used on a building system must comply with the necessary performance criteria of the National Construction Code (NCC), although due to the novel nature of the system, provisions required in order to comply with the NCC cannot be followed. For a system not directly relatable to the NCC, a performance solution must be completed in order to show that it meets the requirements, or a combination of a performance solution and also deemed-to-satisfy criteria is met where applicable. Achieving the NCC's fire performance criteria is particularly difficult to demonstrate when compared to acoustic and structural performance. The fire performance requires the skills of a fire engineer and may involve some of all of the following; a desktop evaluation, computer assisted modelling and physical testing.

One of the main fire design considerations the assessor must give for prefabricated structural systems such as sandwich panels is the connections between adjacent elements as some arrangements provide a pathway for heat and smoke to pass through. The NCC outlines that the combustibility of materials must be tested in accordance with AS 1530.1 (Methods for fire tests on building materials, components and structures – Combustibility test for materials). As many products used in Australia are imported, the correlation between overseas standards and codes do not necessarily align with what is outlined in the NCC, which further complicates the assessment of such materials.

When considering the sandwich panel system as a whole, the NCC refers to two separate Australian Standards, the first being AS 1530.4 (Methods for fire tests on building materials, components and structures – Fire-resistance test of elements on construction). This standard has details in regard to the timeline (given in minutes or hours) for the fire resistance of an element, for the time the element is expected to withstand the exposure to fire and not contribute to the spread of fire to other parts of the building. The second standard of AS 5113 (Fire propagation testing and classification of external walls of buildings) applications where sandwich panels are commonly used, referring to the performance-based solution in the NCC, specifically outlining the spread of fire in the external façade and between buildings.

2.3 Autoclaved Aerated Concrete (AAC)

2.3.1 Description

Autoclaved Aerated Concrete (AAC), also known as Aerated Cellular Concrete or Aircrete, is based on a series of process patents as described by Van Boggelen (2014) which has been used as a building material since the start of the 20th century. A chemical reaction between a specific amount of calcareous and siliceous material is performed to cause aeration in order to produce the lightweight cellular structure, or alternative methods such as mechanical methods to generate the air voids can also be used. When used for structural purposes such as floors, walls and roofs the material is autoclaved using a high-pressure steam-curing process with the end product typically in the form of panels or masonry blocks (Aroni 1993).

2.3.1 Fire Rating

AAC is regarded as a non-combustible building material and with testing through the CSIRO, has achieved Fire Resistance Levels (FRLs) of 60 minutes through to 240 minutes. The factors for the FRL is building application, thickness, density, steel coverage and clear span between support and fixings (CSR Hebel 2006).

2.3.2 Production

In order to produce AAC, fine grading raw materials including silica or quartz sand, lime, cement and aluminium powder are used in a process shown in Figure 2.3. The mineral based aggregates silica and quartz sand can be obtained from broken rocks or granite and can be used in conjunction with fly ash, slag or mine tailings which are then mixed with the lime, cement and finally water to commence the hydration process. The foaming agent is then added which will see the volume increase from 2-5 times the original volume of the mix, depending on the amount of expansion agent is used to get the desired outcome (Hamad 2014). Van Boggelen (2011) states that the best foaming agent used in AAC production worldwide has been found to be aluminium in the form of either powder of pastes. This typically equates to approximately 0.2% to 0.5% of the dry weight and when added to the mixing materials it reacts with the hydroxide of calcium or alkali which releases gas and forms bubbles which results in the increase in volume as the light hydrogen is replaced by the denser air. This sequence is represented in the equation below (Van Boggelen 2011):

$$2Al + 3Ca(OH)_2 + 6H_2O \rightarrow 3CaO \, . \, Al_2O_3 \, . \, 6H_2O + 3H_2$$

Once this uncured material is removed from the mould and cut to the desired length (if applicable), the autoclaving process is undertaken, which cures the concrete in a chamber that exhibits high temperature and high pressure (4-16MPa) for a specified amount of time (8-16 hours). After curing, the AAC segments are finalised and ready for dispatch.

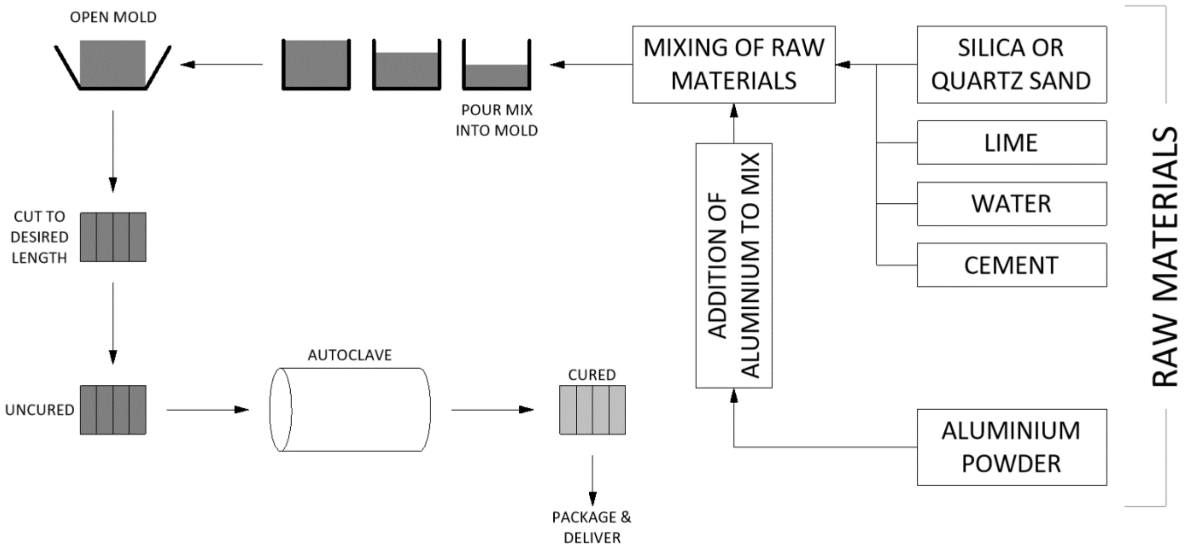

Figure 2.3 - Production Process

2.3.3 Characteristics and Properties

Newman and Choo (2003) have given the typical properties of autoclaved aerated concrete as shown in Table 2.3, with potential for reasonable variation between manufacturers.

Table 2.3 - Typical AAC Properties

Dry Density (kg/m^3)	Compressive Strength (MPa)	Flexural Strength (MPa)	Modulus of Elasticity (E-Value) (GPa)	Thermal Conductivity (3% Moisture (W/mK)
450	3.2	0.65	1.6	0.12
525	4.0	0.75	2.0	0.14
600	4.5	0.85	2.4	0.16
675	6.3	1.00	2.5	0.18
750	7.5	1.25	2.7	0.20

With the density being the main characteristic when specifying a proposed AAC configuration, it has been used in Figure 2.4 to show exactly how the variation in density influences the remaining properties (Narayanan & Ramamurthy 2000).

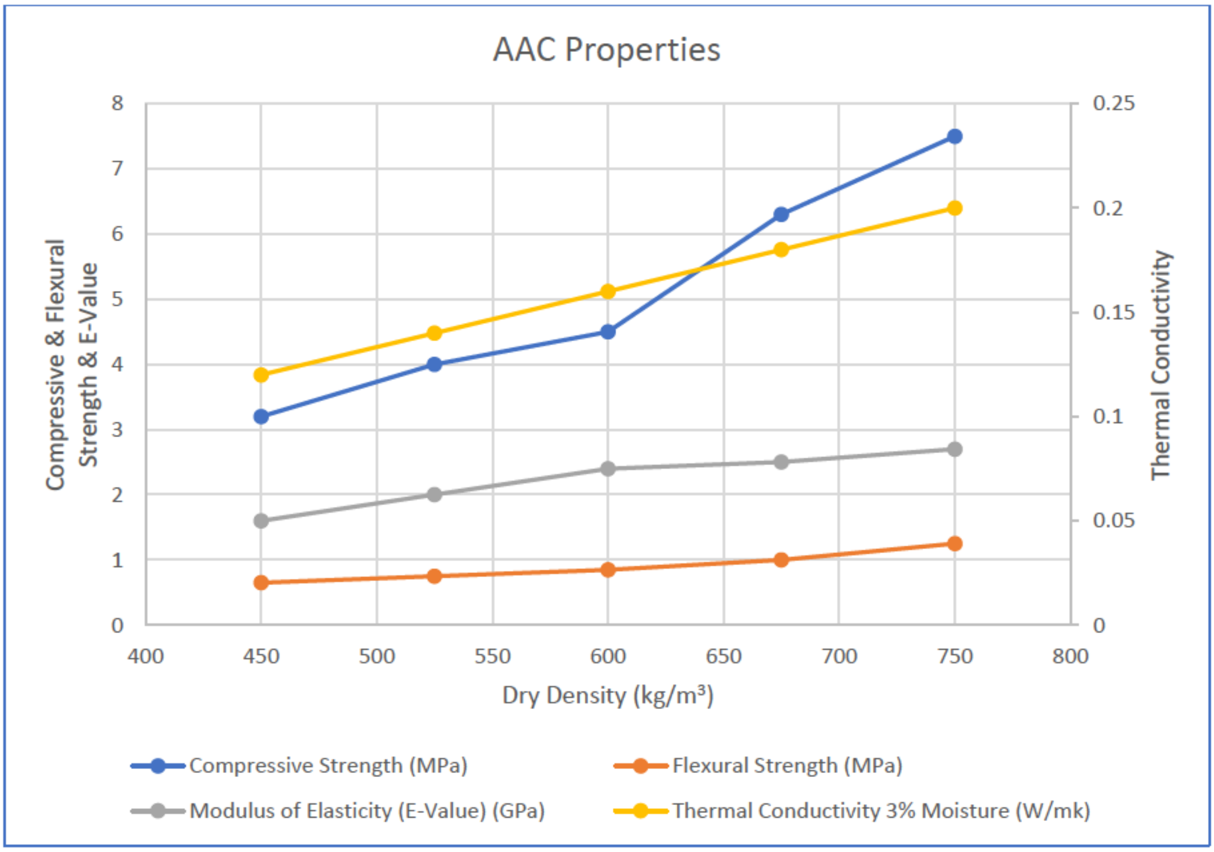

Figure 2.4 - AAC Properties

2.3.3.1 Density

Qu and Zhao (2017) detail how the bulk density of AAC is directly affected by the amount of aerating agent used and the specific gravity of the mix materials. The bulk density has a large amount of variability for the manufacturing process and also has a large influence on many of the important AAC properties including the compressive strength, dry shrinkage and the thermal conductivity (Tada 1986).

2.3.3.2 Compressive Strength

In the case of AAC, the compressive strength refers to the amount of stress that can be carried by the walls of pores, and therefore the configuration in terms of number of pores and the distribution of the pore-size have a direct bearing on this (Qu & Zhao 2017), with Schober (1992) outlining the large influence that the pore structure has on the compressive strength. Aroni (1993) identifies that as the density increases, so too does the compressive strength and other research is in agreeance, with a general linear relationship with density and compressive strength.

The autoclaving process also contributes to higher strength, with no additional curing required due to the high pressure and temperature autoclaving environment. Typical compressive strength values are given in MPa and are shown in Table 2.3 which provides a guide of potential values with the corresponding densities.

2.3.3.3 Tensile and Flexural Strength

Aroni (1993) reports that the direct tensile strength of AAC can vary between 15-35% of the compressive strength discussed in section 2.3.3.2, and also notes that the high degree of variation can be put down to sensitivity of the test conditions. Narayanan and Ramamurthy (2000) feature as similar comparison for the tensile strength and show the flexural strength guide to be 22-27%, with typical values given in MPa as shown in Table 2.3, which provides a guide of potential values with the corresponding densities. An estimation for the modulus of rupture is also given as:

MOR (N/mm^2) = $0.27 + 0.21f_{ct}$

Where:

f_{ct} = Compressive strength of AAC

2.3.3.1 Shear Strength

The shear strength for AAC produce in the form of blocks are commonly specified, though for panel applications (consisting of a rectangular profile such as the configuration of the core material for a stressed-skin panel) the shear strength is often not given by the manufacturers. Mathey (1988) reports the shear strength to be about 1/8th of the compressive strength.

2.3.3.1 Modulus of Elasticity

The modulus of elasticity of AAC is much lower than traditional concrete, with typical values ranging from 1.0 to 8.0 GPa for densities between 400 and 1200 kg/m^3 respectively (Qu & Zhao 2017). Narayanan and Ramamurthy (2000) has summarised the predictive functions for modulus of elasticity for AAC as the following:

$6000(\alpha)^{1.5}\,S$	Where: α = oven-dry density (g/cm^3)
	S = cube compressive strength (kg/cm^2)
$1550\,S^{0.7}$	Where: S = cube compressive strength (kg/cm^2)
$3000\,S_p$	Where: S_p = prism strength (kg/cm^2)
$k\rho(f_c)^{0.5}$	Where: k = constant ranging from 1.5 to 2.0
	ρ = dry density (kg/m^3)
	f_c = compressive strength
$c_1(\rho - c_2)$	Where: c_1 and c_2 = constants
	ρ = dry density (kg/m^3)

Typical modulus of elasticity values is given in GPa and are shown in Table 2.3, which provides a guide of potential values with the corresponding densities.

2.3.3.2 Thermal Conductivity

The thermal conductivity of AAC is said to be anywhere between 10 to 20 times less than conventional concrete, and this strong thermal insulation can be put down to the cellular nature of the product (Bonakdar et al. 2013). Qu and Zhao (2017) describe the contributing variables as predominately the density, along with the moisture content, mix materials and the pore structure. The typical values range from 0.1 to 0.7 W/(m.K) for dry densities between 400 and 1700 kg/m^3 respectively and can be seen in Table 2.3, which provides a guide of potential values with the corresponding densities.

2.3.4 Environmental

Aroni (1993) makes mention of the environmental implications of using AAC as a material and in particular how an 80% pore content reduces the raw materials required in construction which also contributes to a reduction in demolition waste. The consumption of raw materials being low compared to overall volume contributes to the benefits of using AAC along with the ability to use raw material additives such as fly ash and slags to reduce industrial waste and material consumption. Walczak et al. (2015) research findings further detail that the AAC production based on utilizing fly ash has been beneficial in Poland where an energy policy is centred around coal and production. The lower density production has less of an impact on the environment with the ability to deliver a larger quantity of product using the same truck and hence reducing the pollution caused by transportation.

The autoclaving process is a notable step in the process requiring energy to instate the 12 bar of pressure and 190°C required to manufacturer AAC and along with the other production processes combine to have a consumption of 2010MJ/m^3 at a density of 500kg/m^3. During production the by-products (excluding the gaseous emissions) are limited to condensate from the autoclaving, hardened AAC offcuts and surplus unharden AAC mixture and are typically reprocessed in modern production cycles or further developed into marketable materials. Figure 2.5 shows how the manufacturing cycle works in relation to the by-products.

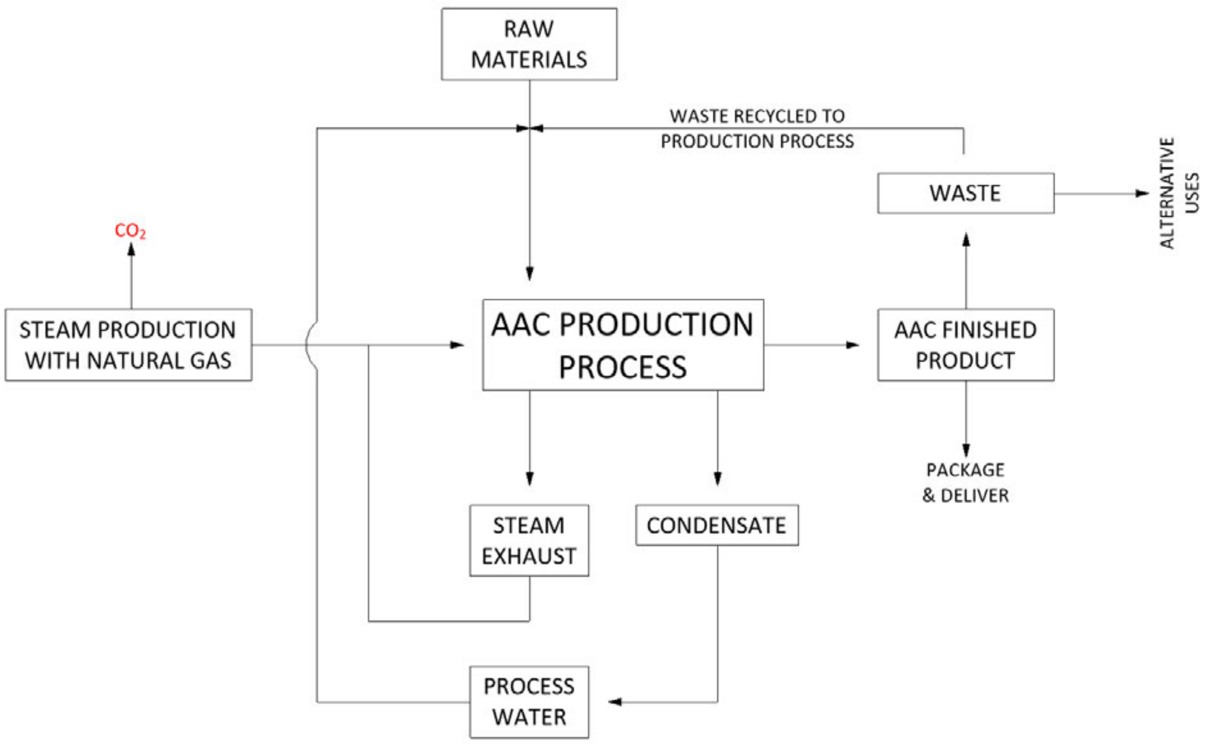

Figure 2.5 - Material Usage Cycle

2.4 Previous Relevant Studies

In commercial terms, the concept of a sandwich panel with an AAC core has not been extensively covered or implemented, but there have been a number of investigations examining the behaviour of similar concepts. The studies of particular relevance include the structural behaviour of composite sandwich panels with reinforced foamed cores and steel faces, which includes testing of both flexural and axial loaded arrangements. The core material in the previous studies are typically standard aerated concrete, with less focus on autoclaved options which can likely be attributed to the expensive facilities, equipment and process required to achieve this material finish when compared to simply foamed concrete. Narayanan and Ramamurthy (2000) outline the differences between autoclaved and non-autoclaved aerated concrete, with a much higher strength due to the more stable form of tobermorite being achieved through the autoclaving process. The drying shrinkage of AAC has also been found to be 20 to 25% of the non-autoclaved equivalent, though generally the aerated concrete offers good functional performance and is comparable in general character to AAC, and hence the studies done in this area are worthwhile for consideration, whilst keeping the aforementioned differences in mind.

Dey et al. (2015) investigated the mechanical response of textile-reinforced aerated concrete sandwich panels with both autoclaved aerated concrete (AAC) and polymeric Fibre-Reinforced aerated concrete (FRAC) as the core materials. The skins layers comprised of a Alkali Resistant Glass (ARG) with the use of a cementitious binder. Three-point bending tests under both static and low-velocity dynamic loading was undertaken in order to be able to evaluate the performance of the arrangements for flexural stiffness, strength and energy absorption capacity. An instrumented drop weight was also used to determine the mechanical properties and how differing impact energy levels influenced them. In comparison to the original unskinned aerated concrete, the addition of skin layers vastly improved the mechanical properties in relation to both the static and impact tests. The flexural strength was increased by up to four times and the skin layers improve the impact resistance. The main factors that the overall performance of the system is said to be reliant on were discussed as the behaviour of the individual components including the core material and thickness, and the interfacial bonding between the skins and core.

With fire safety being a subject of importance around the world, and in particular the severe nature of bushfires experienced in Australia, Nguyen et al. (2018) undertook both a numerical and physical test of the fire resistance of a prefabricated bushfire bunker using aerated concrete panels. Indicating that lightweight aggregate walls maintain a better post-fire structural behaviour when compared to conventional aggregate combination, a 3m x 0.078m 700 kg/m^3 aerated panel with a 0.4mm thick BMT galvanised sheet where combined to create the single panel combination as shown in Figure 2.6.

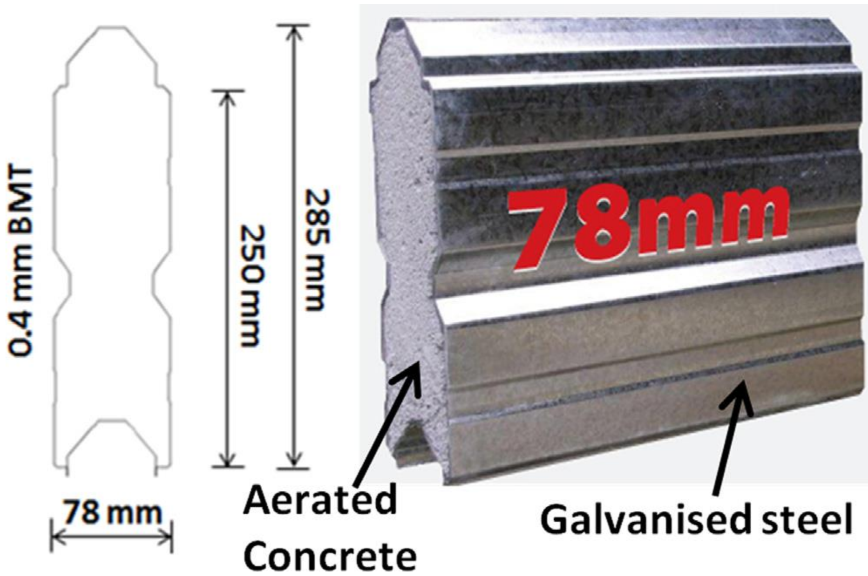

Figure 2.6 - Single Panel Arrangement

The fire testing for bushfire-prone areas is tested against the exposure to a radiation panel of 12.5kW/m^2 to more than 40kW/m^2. The fire resistance level (FRL) adopted by Nguyen et al. (2018) for the external walls in accordance with AS1530.4-2014 was -/30/30. The FRL of an element as outlined in AS1530.4-2014 is given as structural adequacy/integrity/insulation with the appropriate regulatory values (given in minutes) rounded down. The single panel test was run for 30 minutes with the failure criteria being if either of the following occurred:

- The average temperature of the unexposed face of the test specimen surpasses the initial temperature (average temperature on the unexposed face measured less than 5 minutes prior to the test starting) by more than 140^{o}C.
- The temperature at any point on the unexposed face of the test specimen surpasses the initial temperature (average temperature on the unexposed face measured less than 5 minutes prior to the test starting) by more than 140^{o}C.

The test results, along with the computational fluid dynamic (CFD) model developed for the panel show that the arrangement was able to resist the heat penetration with temperature on the unexposed surface not exceeding 100^{o}C for 30 minutes when subjected to the standard fire test given in AS1530.4-2014 and AS3539-2009. As the study focussed particularly on bushfire applications with additional requirements and the principle function of the structure to protect the occupants who will remain in close confines to the panels, two layers of panels were recommended to ensure the human tolerance limit smoke temperature will not be exceeded under such conditions.

Flores-Johnson and Li (2012) investigated the four-point bending characteristics and failure mechanisms of composite sandwich panels with corrugated steel faces with a combination of plain or fibre-reinforced foamed concrete core, as shown in Figure 2.7. Testing of the 1000kg/m^3 foamed concrete included a uniaxial compression, indentation test and uniaxial tensile test. The outcomes included the fibre reinforced concrete sample demonstrating higher compressive strength and an increase of the tensile modulus and strength of 168.4% and 558.1% respectively. During the four-point bending testing, shear failure under the loading points was evident whilst still being able to perform its primary function of separating the face sheets. Further increases in loading lead to the sheet skins wrinkling and debonding between the core and skins and slippage of the bottom sheet was also evident. To better understand the relationship between the face/core bonding and fastening on the four-point bending response of the sandwich panels, finite element examination was undertaken. It was found that the face/core bonding is critical for the structural performance whilst the fastening was found to have an insignificant contribution.

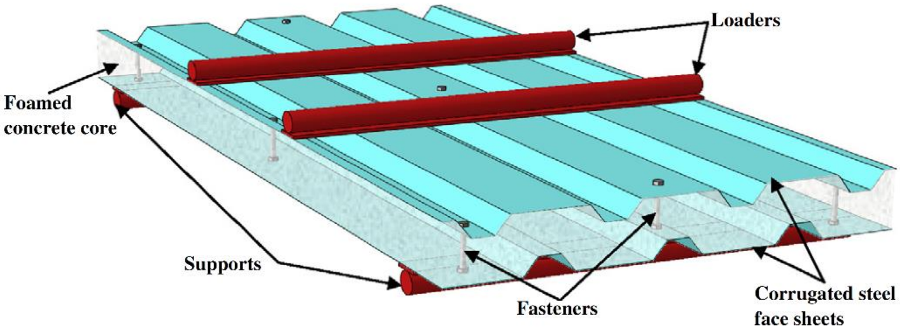

Figure 2.7 - Panel Arrangement

2.5 Summary of Literature Review Findings

A number of important findings were made as a result of the literature review presented throughout chapter 2 which solidified the need to embark on this study whilst uncovering new information that will contribute to the direction in which subsequent chapters of this dissertation will head.

A summary of the findings can be given as:

- Stressed-skin (sandwich) panels are an effective use of materials for many applications as a result of good performance and inherit characteristics.
- The typical failure traits are well documented.
- There are significant safety issues with some existing core materials, in particular flammability and toxicity.
- Substantial evaluation, testing and simulation data is required for a new composite structure (such as the system proposed in this dissertation) to adhere to the necessary code requirements in Australia for fire resistant levels.
- The production methods for AAC is well established, along with most data for the material properties.
- Shear strength for AAC in the form of panel arrangements is not well documented in comparison to block form.
- The characteristics of AAC generally address the main disadvantages with typical core materials such as foam.
- There is not a vast amount of previous research completed using AAC as the core material for sandwich panel applications, underlining the importance of the proposed research.

3 Methodology

This chapter is intended to provide a short overview of the methodology that will commence in more detail in the following chapters, which lead up to the results provided and discussed in chapter 8.

Firstly, testing of Hebel AAC panels with different skin materials will be undertaken via four-point bending tests, to gather relevant data regarding deflection and ultimate failures. With this data, an analytical analysis of the testing results will be undertaken in an attempt to replicate the experimental findings, both in terms of deflection and ultimate failure of each structure. Any differences will be identified, and assessments will be made (if necessary) to ensure that the remaining predictive analysis can achieve a better degree of accuracy.

The different potential applications of use in a building system will be investigated, and the criteria established from the relevant Australian standards and codes to use for the eventual design of the stressed-skin AAC core panels. The analytical processes developed after the testing analysis will be used and adapted as necessary to design structural stressed-skin panels for the highlighted application against the criteria found. This will allow span tables to be populated with the design data and hence develop a greater understanding of the factors involved, their importance and the most efficient, optimised solutions presented for consideration.

4 Experimental Investigation

4.1 Introduction

Experimental testing was undertaken in order to gain further understanding as to how Hebel AAC panels perform in a real-world testing environment, with different skin materials to help give some comparison between the variables and to provide direction for the ongoing developments and conclusions to be made in this report. The initial testing took place in February which was followed by additional testing taking place in June, which was influenced by the observations and outcomes established from the initial testing. The data retrieved from each test allows for ensuing comparison and analysis against theoretically found predictions for each respective arrangement in order to further extrapolate data into other potential areas of interest in a valid way.

4.2 Testing Description (February)

On the 21st of February 2019, USQ Structural Testing Services conducted a load test and subsequent report (report ID number STS-19-013) for three test samples which consisted of a typical Hebel panel with varying skin configurations to the two main longitudinal panel faces.

The Hebel profile consisted of a 2550 long x 600 wide x 75mm thick Hebel panel comprising of 4mm centrally located mesh reinforcement spanning across the panel with 3/4mm longitudinally reinforcement, placed centrally and 60mm in from each side. Each test consisted of the same Hebel size and specifications and the skin configuration became the changing variable.

Test sample 1 was used as a control and had no skin facings, whereas test sample 2 had a fibreglass (GFRP) skin to each side and the final test sample number 3 consisted of steel skins (Stramit CustFish 801-900G 6B 0.55 Zincalume). Figure 4.1, Figure 4.2 and Figure 4.3 show a section view of each test sample, showing the dimensions for thickness and width, along with the reinforcement specifications and skin arrangements.

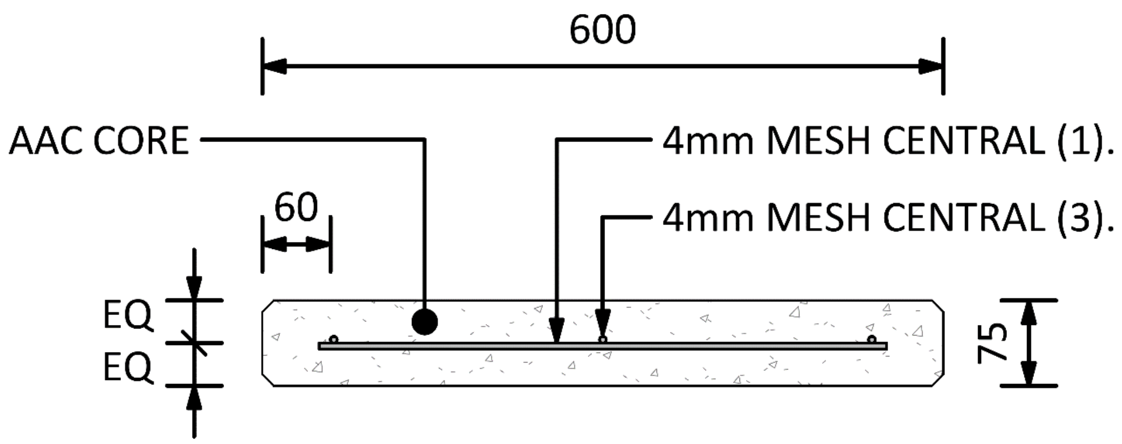

Figure 4.1 - Section View of Test Sample 1

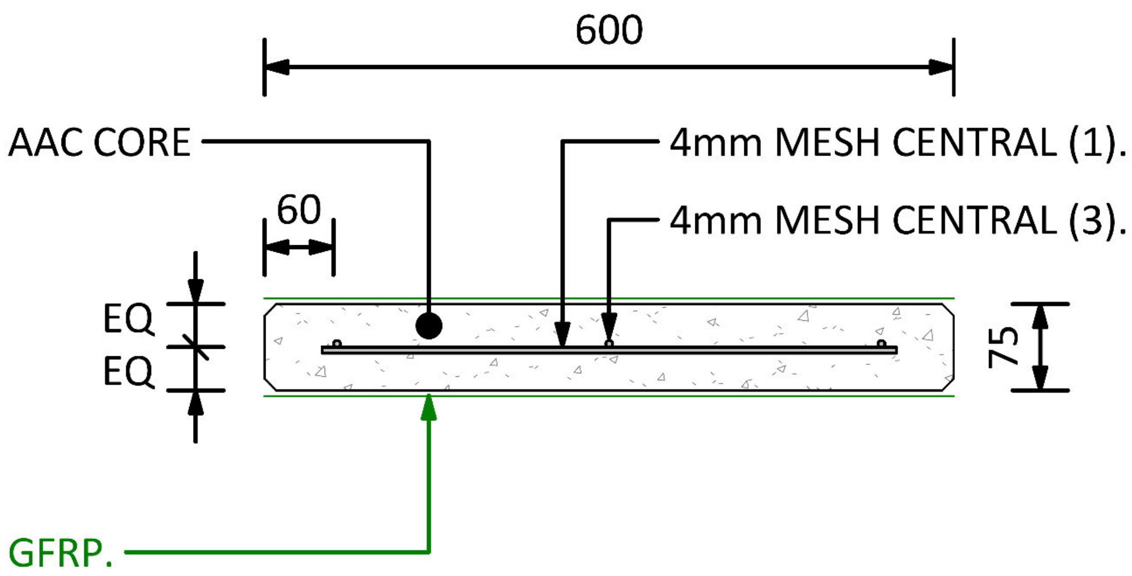

Figure 4.2 - Section View of Test Sample 2

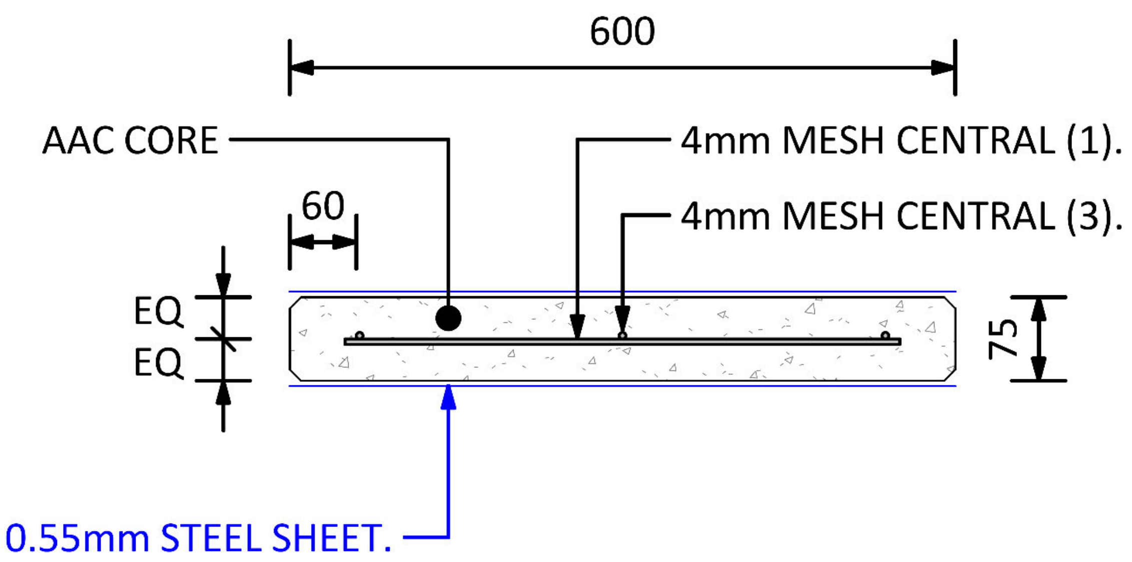

Figure 4.3 - Section View of Test Sample 3

The objective of the test was to observe and compare the behaviour of the samples under load testing and to retrieve usable numerical data in regard to increasing load versus mid-span deflection. Each test sample was set up on the test machine where it was spanning a consistent 2000mm for all the tests. The four-point bending testing arrangement can be seen in Figure 4.4 and the data was then retrieved through a testing facility where the load was applied through a load frame with 1000mm between the inside of the load points, and a laser measurement was used to plot the displacement at a location central to the panel.

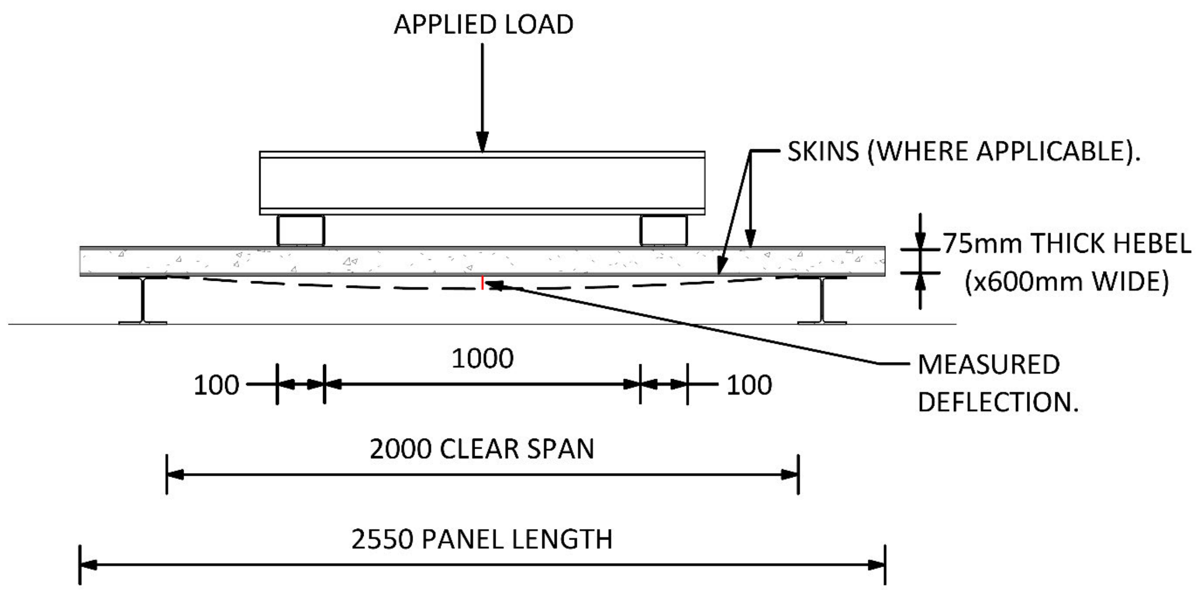

Figure 4.4 - Testing Arrangement

4.2.1 Test Sample 1

4.2.1.1 75mm Hebel Panel – No Skins

The first test sample was used as a control test with no skin material to allow for a direct comparison against the subsequent tests which had varying skin materials bonded to each side of the Hebel panel. This sample failed at a peak load of 1,583N and as the load increased there was some visible vertical orientated cracking to the tension face (underside of the panel as shown in Figure 4.5) before an eventual complete brittle bending failure across the entire width of the panel towards to the centre of the span as can be seen in Figure 4.6.

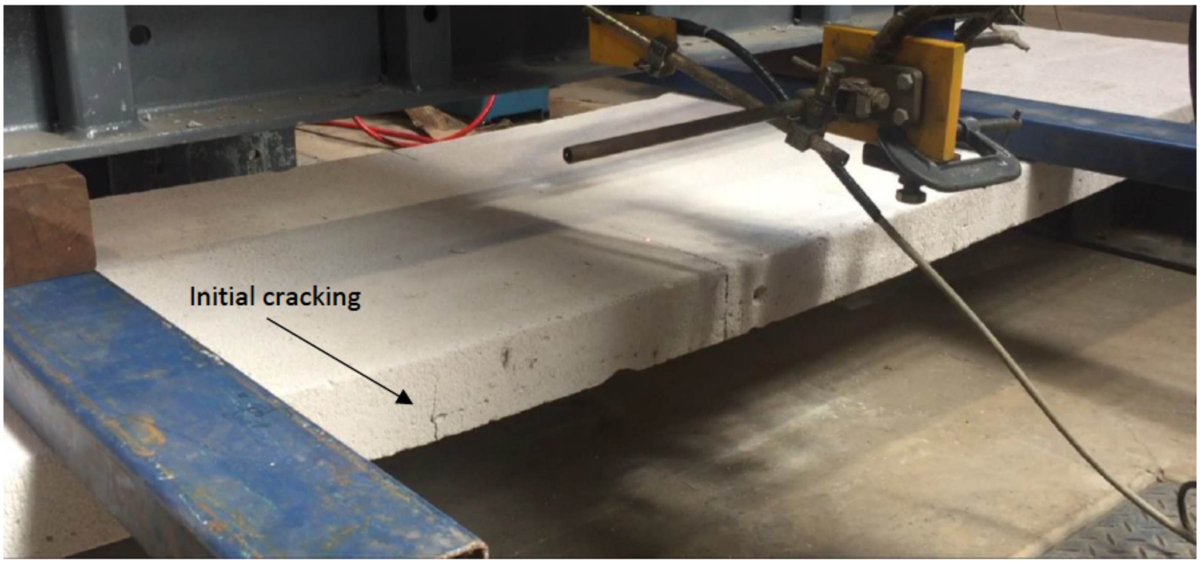

Figure 4.5 - Test Sample 1 Initial Cracking

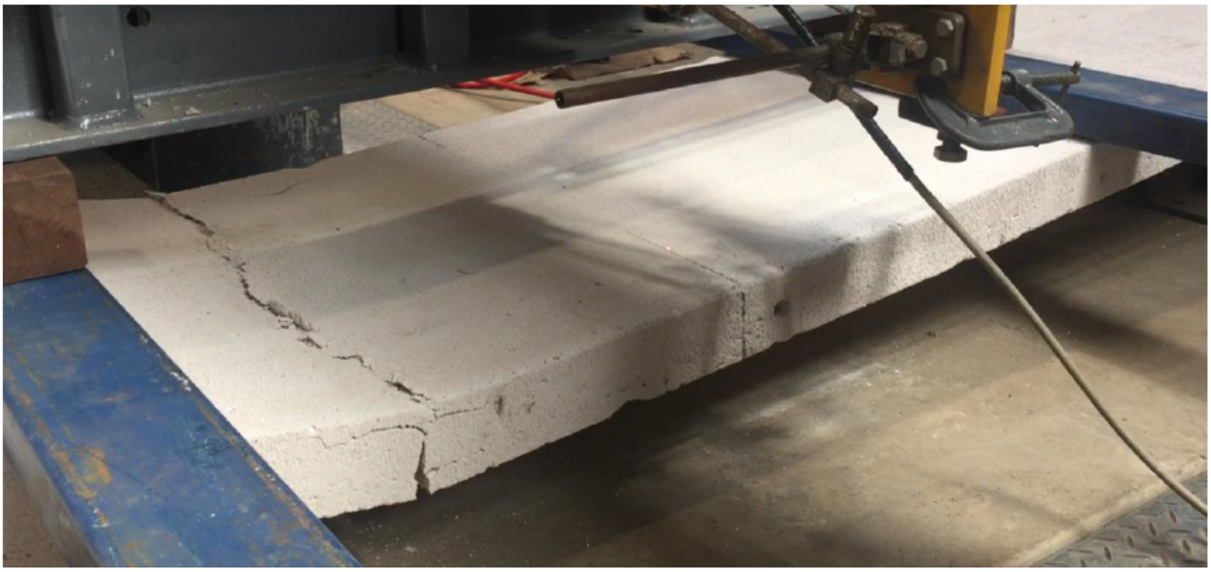

Figure 4.6 - Test Sample 1 Failure

4.2.2 Test Sample 2

4.2.2.1 75mm Hebel Panel – Fibreglass (GFRP) skins

The second test sample used fibreglass (GFRP) skins bonded to each side of the Hebel panel and this sample failed at a peak load of 10,440 N. It was observed that as the load increased there was some visible horizontal orientated cracking to the tension face (underside of the panel as shown in Figure 4.7) towards the centre of the span which was followed by a shear failure of the Hebel at a location adjacent to the loading area as can be seen in Figure 4.8.

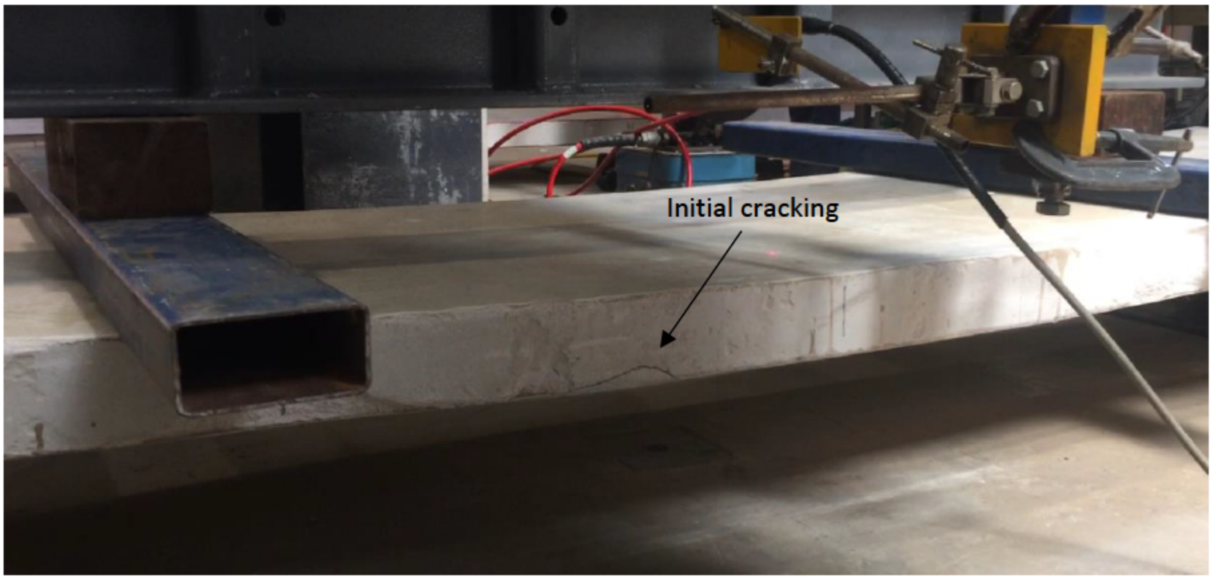

Figure 4.7 - Test Sample 2 Initial Cracking

Figure 4.8 - Test Sample 2 Failure

4.2.3 Test Sample 3

4.2.3.1 75mm Hebel Panel – Steel skins

The third test sample used steel skins bonded to each side of the Hebel panel and again the same test was repeated. This sample failed at a peak load of 11,460 N and as the load increased there was some cracking originating at the tension face (underside of the panel) at a similar location to where the second test sample experienced complete failure, adjacent to the loading area. This test sample did not experience complete collapsing failure but was terminated after the steel skin to the tension face (underside of the panel) became detached from the Hebel AAC core after small signs of cracking which was then followed by a complete shear crack occurring in the core. The core damage and skin detachment can be seen in Figure 4.9.

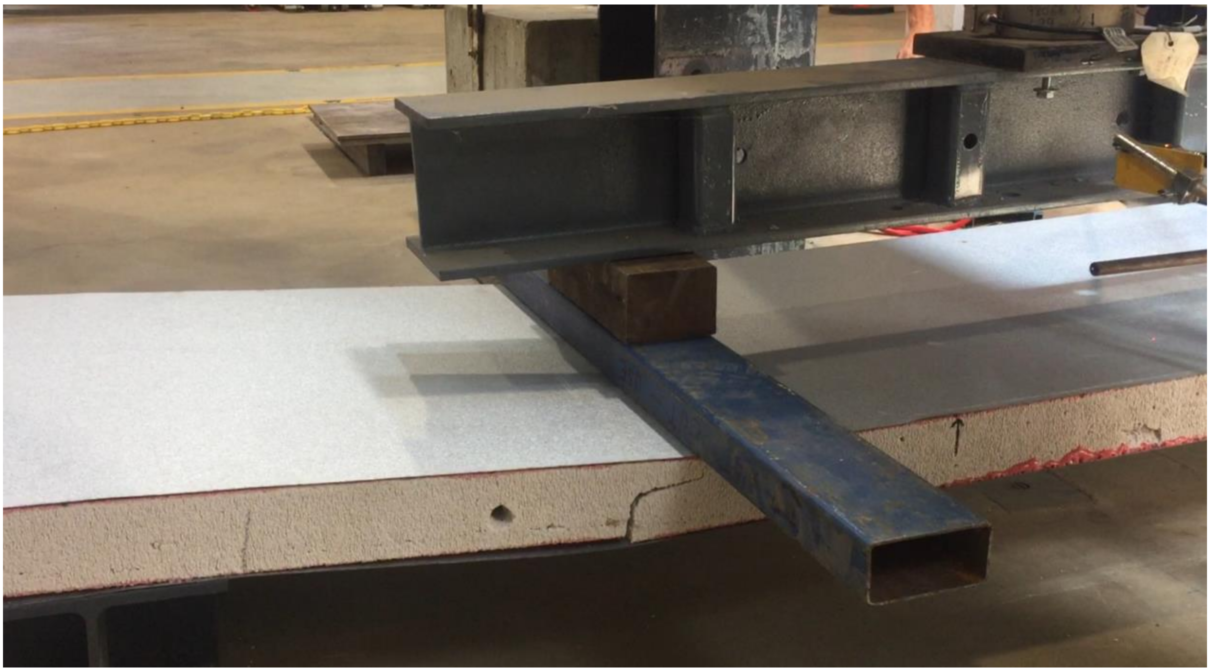

Figure 4.9 - Test Sample 3 Damage

4.2.4 Test Results (February)

Table 4.1 allows a direct comparison between the peak load (in Newtons) at which each sample test was ceased due to failure or debonding. Other information retrieved from the test included the corresponding deflection, comparison against the control (test sample 1) and a comparison for a specific defection amount of 10mm.

Table 4.1 - Testing Load vs Deflection

Test Sample	Peak Load (N)	Deflection at Peak Load (mm)	Increase Peak Load Over Control	Applied Load at 10mm Deflection (N)	Increase Load Over Control at 10mm Deflection
1	1583	44.79	N/A	718	N/A
2	10440	38.19	559%	4301	499%
3	11460	13.32	624%	9395	1208%

Figure 4.10 plots the entire load versus deflection relationship for the test samples to allow easy comparison between the different arrangements.

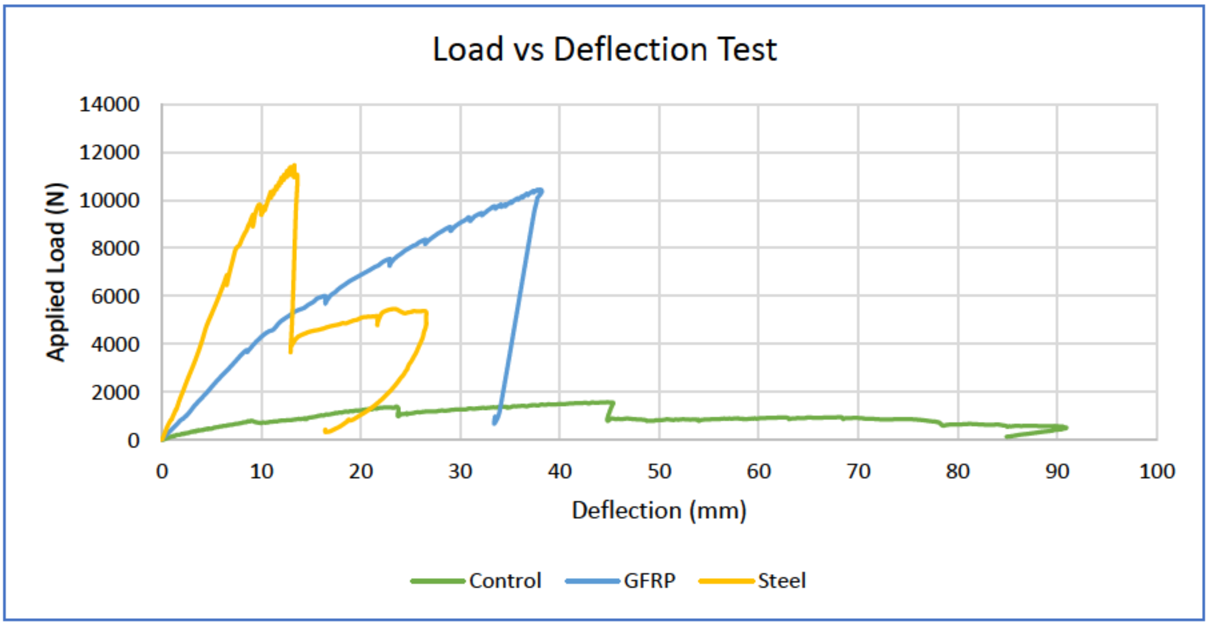

Figure 4.10 – Test Samples 1, 2, 3 Load vs Deflection

4.3 Testing Description (June)

On the 4th of February 2019, USQ Structural Testing Services conducted a load test and subsequent report (report ID number STS-19-072) for one test sample which consisted of a Hebel panel with steel skin facings (Stramit CustFish 801-900G 6B 0.55 Zincalume) to the two main longitudinal panel faces and lapping down the sides and fixed into the Hebel panel, as shown in Figure 4.11. The chosen arrangement stemmed from the February testing results indicating that the steel skin facings performed well under the conditions in comparison to the GFRP facing, and was therefore implemented again in this test (Test Sample 4) to further validate and collect data for further understanding, analysis and extrapolation of data.

Figure 4.11 - Test Sample 4 Arrangement

The Hebel profile consisted of a 6000 long x 600 wide x 150mm thick Hebel panel comprising of 35/Ø5mm centrally located reinforcement spanning across the panel with 8/Ø6mm longitudinally reinforcement as shown in Figure 4.12.

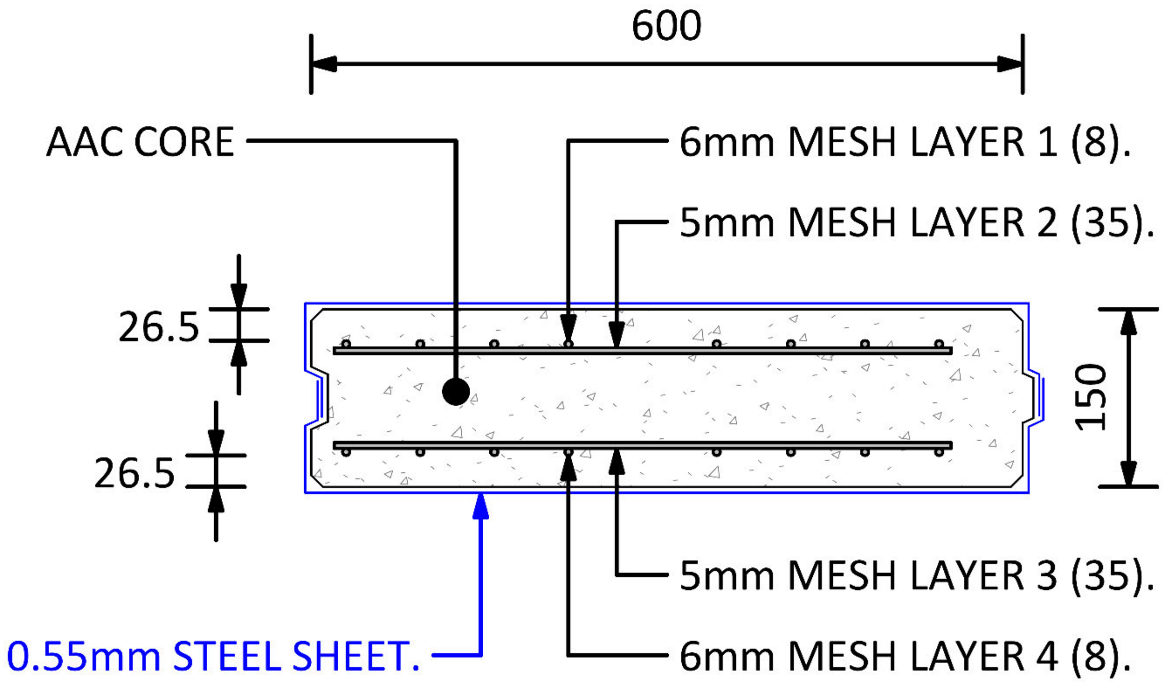

Figure 4.12 - Section View of Test Sample 4

The objective of the test was similar to the tests carried out in February, although the different panel configuration gave particular focus on a larger span (and consequently thicker core) in order to determine the concept's suitability for other potential applications. The test sample was set up on the test machine where it is was spanning 5340mm. The four-point bending testing arrangement can be seen in Figure 4.13 and the data was then retrieved through a testing facility where the load was applied through a load frame with 1400mm between the inside of the supports, and a laser measurement was used to plot the displacement at a location central to the panel.

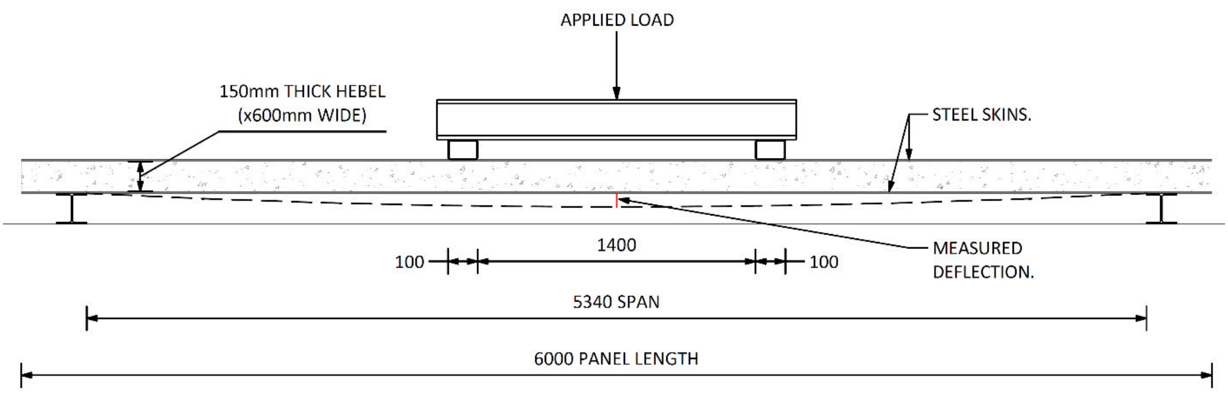

Figure 4.13 - Testing Arrangement

4.3.1 Test Sample 4

4.3.1.1 150mm Hebel Panel – Steel skins

The fourth test sample used steel skins bonded to each side of the Hebel panel and again the same test was repeated. The test for this sample was ceased at a peak load of 24,186 N and as the load increased there was evidence of the top sheet wrinkling, particularly the area parallel to the line of loading with is depicted in Figure 4.14. Sheet wrinkling was also evident in between the loading points once the deflection became significant. Due to the sheet wrinkling, complete failure of the core was not established under the loading conditions.

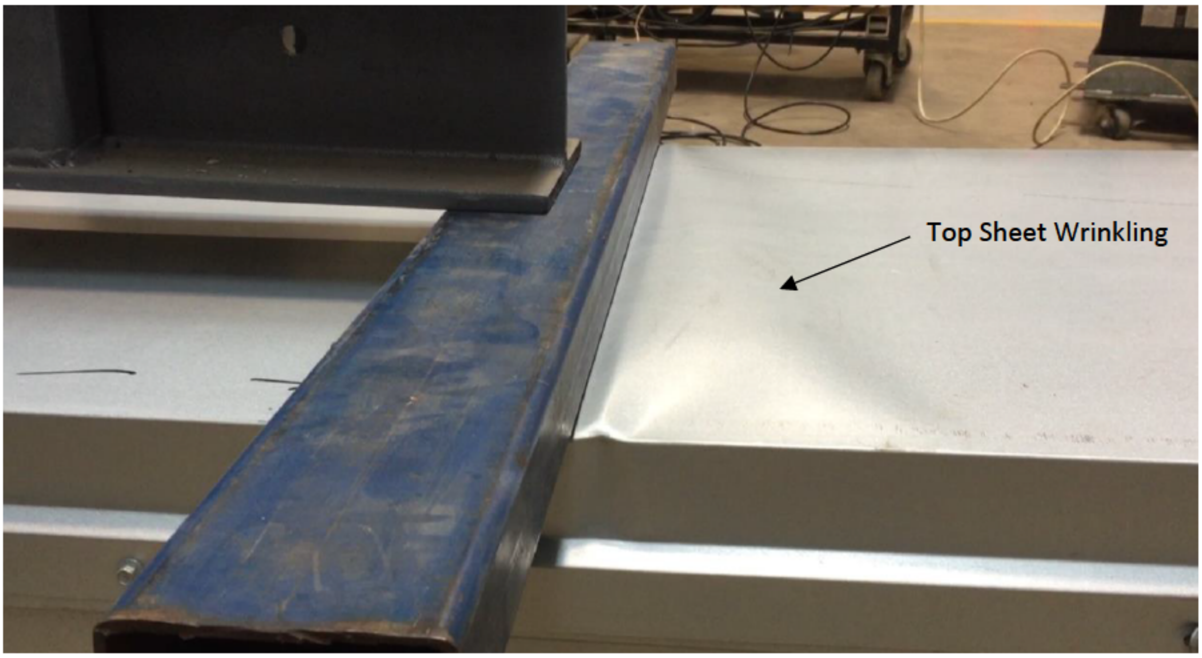

Figure 4.14 - Test Sample 4 Damage

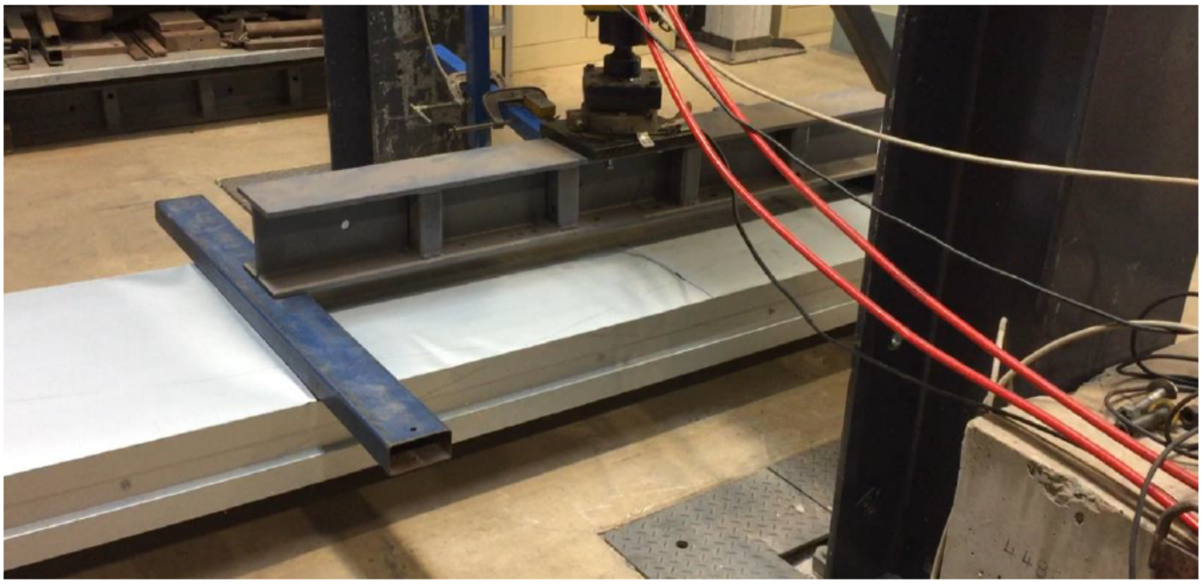

Figure 4.15 - Test Sample 4 Damage

4.3.1 Test Results (June)

Figure 4.16 shows the full test load versus deflection data for test sample 4, with a peak load of 24,186 N resulting in 58.90mm as the corresponding deflection given in Table 4.2.

Table 4.2 - Testing Load vs Deflection

Test Sample	Peak Load (N)	Deflection at Peak Load (mm)
4	24186	58.90

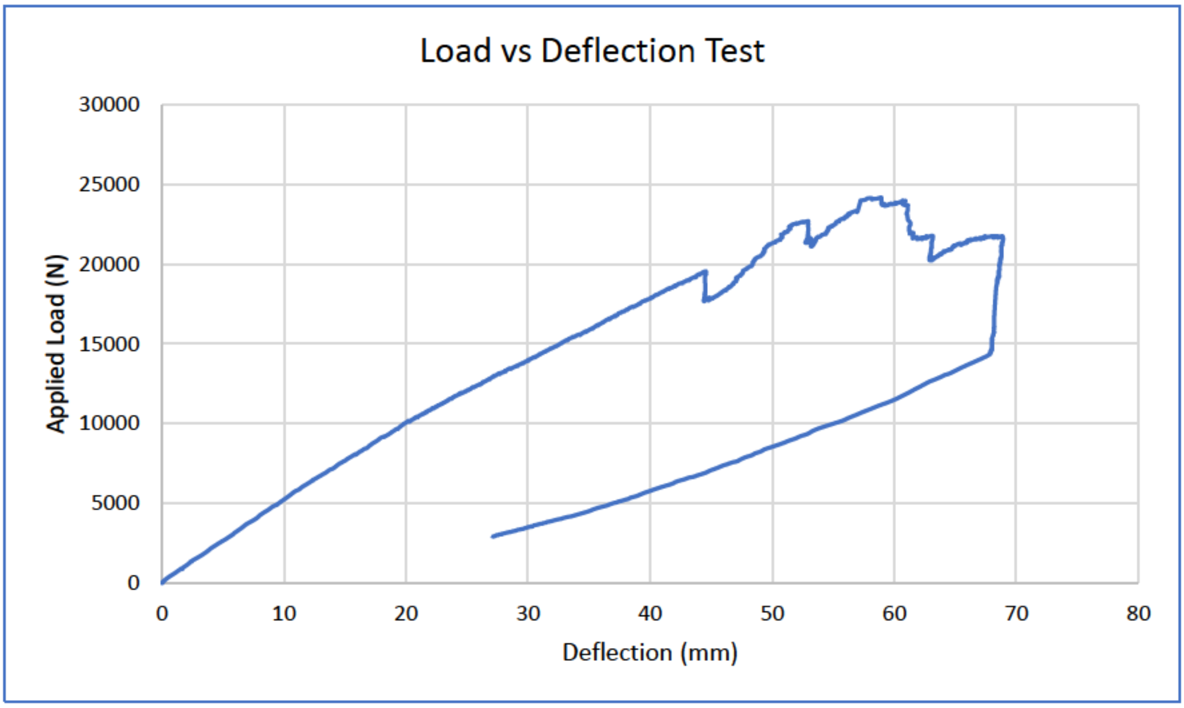

Figure 4.16 – Test Sample 4 Load vs Deflection

4.4 Test Discussion Points

4.4.1 Performance

It can be seen that by lining the Hebel panel with GFRP or steel skins, the performance in terms of relative deflection and ultimate performance is significantly improved when compared to the control panel. This was reflected in the peak loading that was achieved, with test samples 2 and 3 showing a 559% and 624% increase in load carrying ability respectively when compared to the control test sample 1.

To further evaluate the performance improvement, 10mm of deflection was chosen as a comparison point between all of the samples, as this was achieved well within the range prior to any significant failures occurring and therefore allows a fair and direct comparison in performance. The GFRP was able to carry a load increase of 499% when compared to the control sample and the steel skinned sample was able to carry an additional 1208% load for the same corresponding deflection of the control test sample.

When comparing the skin options of GFRP and steel directly against each other, the ultimate maximum load carrying capacities produced similar results before shear failure of the core (10,440N and 11,460N respectively). The major difference between the two options was evidently the rate of deflection, with the GFRP test sample defecting consistently around three times more than that of the steel option, when carrying the same applied load. This suggested that the shear resistance is provided by the core material and the skin material top and bottom (separated by the core) is the largest contributor to that way in which each sample resists bending.

4.4.2 Failures

The testing results and failure modes are consistent with the expectations provided in section 2.1.4 of the literature review for the test sample with skin materials present, meaning that the introduction of a new core material behaves in a similar manner to that of previous findings and literature. Test sample 1, without the presence of skin materials, was seen to fail in bending at the tension face which is an expected mode of failure with concrete materials being poor in carrying tensile forces. Other observations from the test also indicate areas that require more refinement, with the potential of using different configurations of the GFRP make-up that have more tensile strength in order to get better performance results in the form of reduced deflection.

With the steel skinned sample in particular, the bonding between the skins and panel is also an area of potential improvement, as the separation resulted in a premature ending to the test and therefore the full potential of the steel skinned option has not been fully understood or explored. The adhesive used to bond the steel skin and Hebel core was a Pacific Urethanes product named UREPAC Bond 10 60 TX and is a single component adhesive based on polyether polyol and modified MDI isocyanate which cures using the moisture in the atmosphere to cure the paste-like product (Pacific Urethanes Pty Ltd 2019). The bonding relationship is an important component of the sandwich arrangement and although noting any findings in regard to this factor will be done, it is not the main focus of this study. A change of the bonding material, process or the specifications of the Hebel AAC make-up may result in a better structural outcome which would require further research and testing to optimise the performance. The point at which the bond is separated and fails is seen to occur once the test sample has experienced significant deflection from its original position, and hence debonding is an expected consequence of the significant movement between the core and skins.

Relating this to the structural requirements of the panel, the point at which debonding becomes critical would be unlikely to be experienced more often than not. This is due to the serviceability limitations placed on each individual criterion given in AS1170.0 (Structural Design Actions Part 0: General Principles) and hence a large amount of deflection would not be permissible, whereas in the testing procedures a much larger load and deflection is experienced to find the limits of each sample. Therefore, although this component of the process (bonding) has the potential to be improved and refined in future investigations, it is not likely to limit the potential uses of the product and will not be further investigated in this report.

Other consistent failures were seen to occur locally at the point of concentrated loading, with both shear failure of the core and wrinkling of the steel face sheets occurring. This is a limitation of the testing arrangement as once the applied load reaches a significant magnitude the localised failure inevitably occurs. The data gathered remains useful and as most realistic potential applications for this product are loaded in a more uniform way and designs are typically based off a pressure (kPa) load which does shift the importance away from localised failures directly attributable to the nature in which the sample was line loaded.

4.4.3 Outcomes

The results for the testing program give a clear conclusion that by using either GFRP or steel skin facings, there is a significant improvement in performance when compared to a standard Hebel panel of the same configuration. The results confirm the obvious load carrying benefits of using a sandwich panel arrangement and therefore provides justification for further research and design work in order to be able to optimise the efficiency of stressed-skin panels with an autoclaved aerated concrete core. It was observed (and verified by the data) that typically the skin material provides the bending resistance for the composite structure and the core material carries and supports the shear forces developed.

This dissertation will focus on steel as the desired skin material, with the quality of results of the steel skinned example far superior to that of the GFRP test sample, particularly when considering limiting deflection. Steel skins are also more commercially readily available and there are many different options for specifications with properties that are well known and understood from a structural perspective.

These factors are also partnered with use of steel skins for many existing sandwich panels and other common systems, which makes the steel skinned AAC core panel an ideal concept moving forward in this study. The industry, in particular architects and their clients wanting a high quality, aesthetic finish, will be more inclined to accept a system that has the flexibility to achieve a more easily customisable finish and achieve a similar result to the building systems that they are familiar with and are current implemented. This material would also contribute to benefits in construction time and labour costs, with the ability to omit the need for cladding systems to be installed on site if the correct finish can be achieved and incorporated into the panel system itself.

The variation of core thicknesses and span lengths explored in the test samples show that the concept has the potential to work in many potential applications, with test sample 3 showing how the 75mm core performs over a small span and test sample 4 having the ability to span a significant distance with the 150mm core. As a result, the concept has flexibility in its available options which will allow it to be refined to suit the specific requirements that may be experienced in different building applications such as the ones explored in chapter 6 of this dissertation.

5 Testing Analytical Analysis

5.1 Introduction

The analytical analysis to be undertaken in this report will be broken into two phases which aims to understand and also predict the capabilities of the concept.

The first phase of the analysis entails the calculation of predicted characteristics and performance that directly relate to the arrangements provided for the test samples, with the aim of being able to replicate and validate the theoretical data against the known data obtained via the test results. This will be undertaken in section 5 of this report.

Once complete, section 7 of this report will use the learnings from the analysis of the test results and manipulate the processes as required, in order to extrapolate the current knowledge of the system to analyse other arrangements outside the testing regime. The results of which can be used to understand the effect that the variation in core thickness, core density, skin thickness and span has on the system from a performance perspective. The analytical analysis will consider the strength and serviceability conditions in order to produce span tables that can then be used to determine outcomes and comment on the applicability of market opportunities for the AAC core, steel stressed-skin panel concept.

5.2 Testing Analysis

The testing undertaken for this report gives numerical data for the deflection characteristics and also a visual depiction of what the failure mode was for each test and also the applied load at the corresponding point of failure. This data will allow a direct comparison to the analytical calculations for both strength and serviceability conditions, highlighting the differences between the predicted and actual characteristics of each relevant test sample.

5.2.1 Serviceability

Figure 5.1 shows the deflected shape that correlates with the static loading condition that a beam experiences during a four-point bending test, such as the one used in the testing undertaking in section 4 of this report.

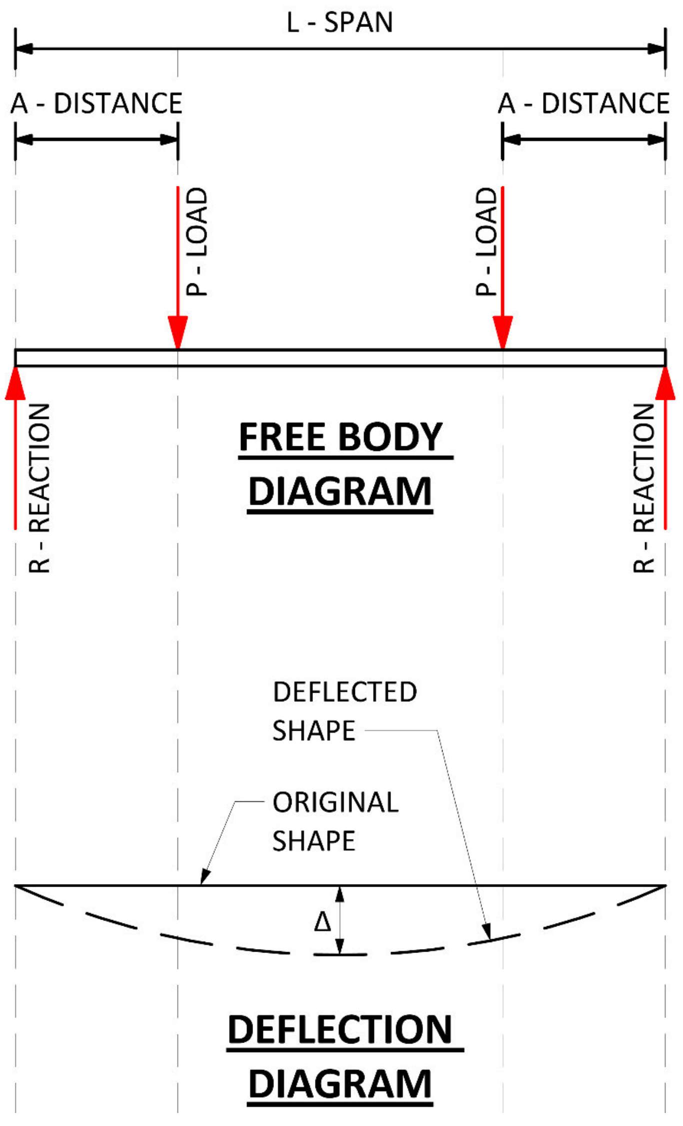

Figure 5.1 – Four-Point Bending Test Deflection Diagram

The relationship between the applied load, flexural rigidity, span and geometric arrangement give the following expression:

Deflection (maximum at center of span)$\Delta = \frac{\mathbf{PA}}{\mathbf{24EI}}(3L^2 - 4A^2)$

Where:

$\Delta = \text{Deflection (m)}$

$\text{P} = \text{Applied Load (kN)}$

$\text{A} = \text{Distance From Support to Applied Load (m)}$

$\text{EI} = \text{Flexural Rigidity } (\text{kN}/\text{m}^2)$

$\text{L} = \text{Span (m)}$

The flexural rigidity (EI) can be found by summing the contribution from each layer within the section of the panel, using the dimensional and material properties to find the total EI value for the section.

The theoretical EI value for test samples 3 and 4 can then be found and compared to the actual EI value calculated from a known point from the test data as shown in Table 5.1. Refer Appendix B for the detailed calculations for the flexural rigidity for test samples 3 and 4.

Table 5.1 - Flexural Rigidity

Test Sample Number	Theoretical EI Value (kN/m^2)	Test EI Value (kN/m^2)
3	222	120
4	1200	1445

The theoretical and test measured load versus deflection graphs shows the degree of agreeance between the predicted and actual outcomes of the testing. This can be seen in Figure 5.2 and Figure 5.3 for test samples 3 and 4 respectively.

5.2.1.1 Test Sample 3

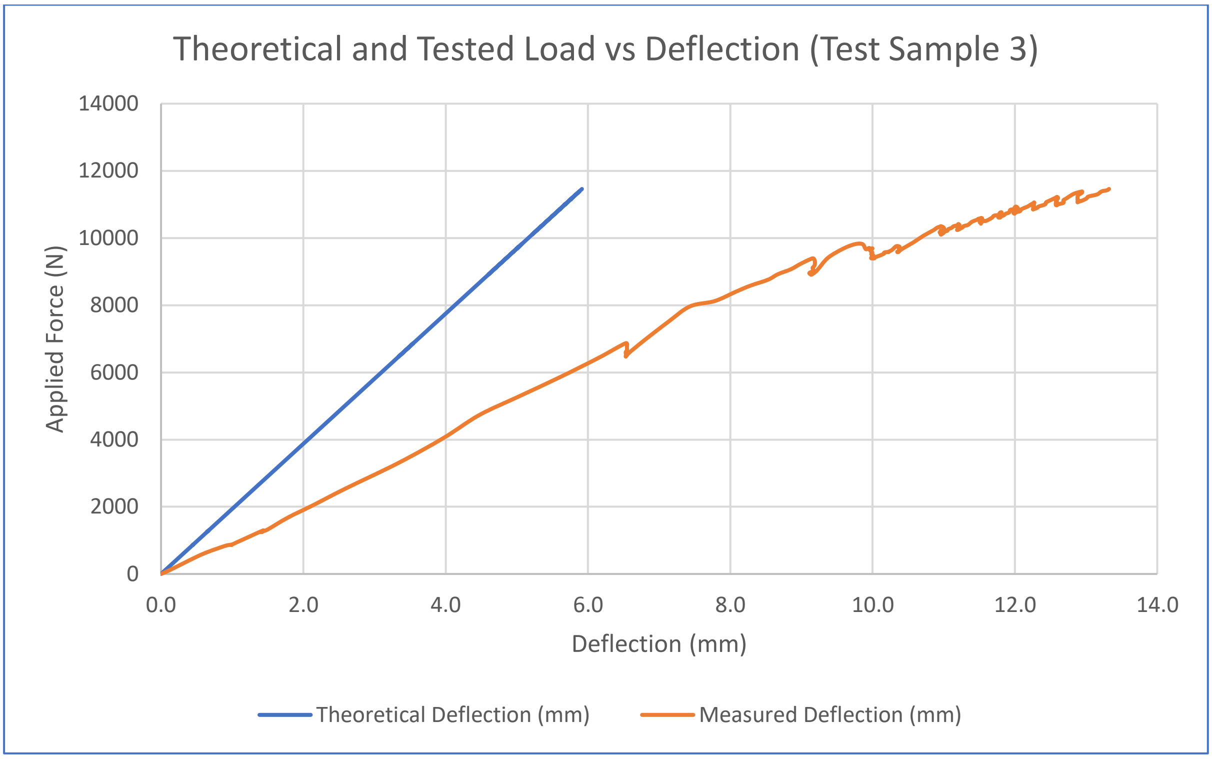

Figure 5.2 - Theoretical and Tested Load vs Deflection (Test Sample 3)

5.2.1.2 Test Sample 4

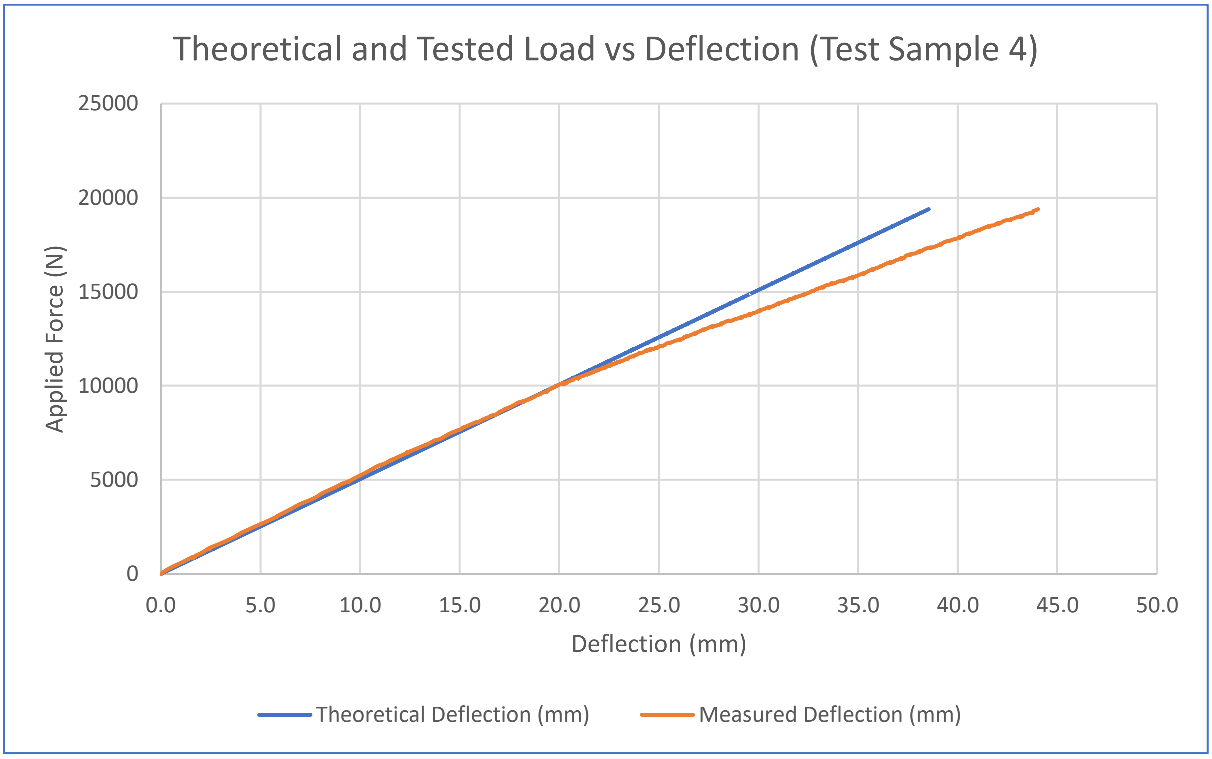

Figure 5.3 - Theoretical and Tested Load vs Deflection (Test Sample 4)

5.2.1.3 Discussion

It can be seen that for the arrangement given in test sample 3, the measured deflection is typically more than the theoretically calculated deflection and isn't an accurate representation of the test sample. This can be put down to the poor bonding between the skin and core materials, which can be seen on the graph by the inconsistent readings and jittering data. As the bond was poor, there was evidence of some premature cracking in the core structure, which would be limited or even prevented if appropriate bonding was achieved. This observation would go some way into explaining the deflection results as it is also known that the skin material is the largest contributing factor towards the bending resistance and hence a poor bond will inhibit the system's ability to resist bending. Therefore, the test sample did not perform to the anticipated level as a result of some of the construction imperfections, which was acknowledged, and an attempt was made to address the issues for the construction of test sample 4.

Test sample 4 showed high amount of agreeance throughout the entire data set with a particularly strong representation between an applied load of 0 and 10kN. In comparison to test sample 3, the construction techniques used in test sample 4 gave a more consistent data set to compare the theoretical predictions against.

5.2.1.4 Outcomes

Therefore, as demonstrated by test sample 4, the analytical method presented in this section of the report (which was used to calculate the deflection) is a valid method for predicting the deflection for an AAC core, steel stressed-skin panel in a four-point bending test. Using this method, there is an assumption that the bond between the skin materials and core is developed sufficiently in order to carry the load effectively.

Further testing would be ideal to gather more data to validate, although the success of the data retrieved in test sample 4 in comparison to the numerical analysis gives some justification for the accuracy of the approach. This will allow further manipulation of the analysis in order to be able to predict the deflection during a more realistic uniformly distributed loading arrangement that will be experienced during real-world applications.

5.2.2 Strength

Figure 5.4 depicts the shear and bending moment diagrams that correlate with the static loading condition that a beam experiences during a four-point bending test, which has associated typical equations which will allow approximate calculation of the forces experienced during the testing.

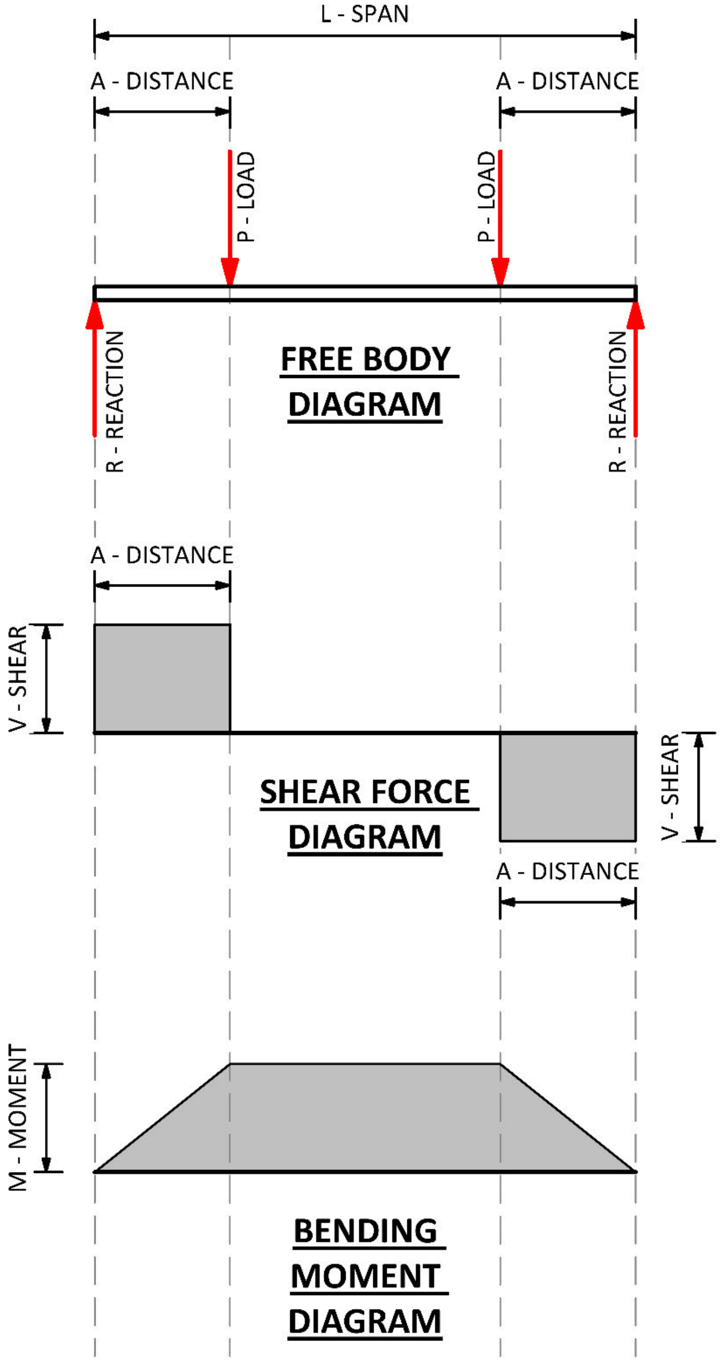

Figure 5.4 – Four-Point Bending Test Force Diagrams

The force diagrams, along with the established equations reflect the conditions seen in the four-point bending test, will allow further understanding of the stresses that the test samples were subject to during the test and at the eventual failure point.

$$R = V$$

$$V = P$$

$$M = PA$$

Where:

$R =$ Reaction at support (kN)

$V =$ Shear Force (kN)

$M =$ Moment Force (kN.m)

$P =$ Applied Load (kN)

$A =$ Distance From Support to Applied Load (m)

Table 5.2 exhibits the approximate forces experienced by each test sample at the point of failure, to allow direct comparison against the theoretical failure points which are found. Noting that test sample 2 has been omitted from future the analysis based on the outcomes given in section 4.4.3 of this report.

Table 5.2 - Test Sample Failure Forces

Test Sample Number	Force At Point of Failure		
	Reaction – R (kN)	Shear – V (kN)	Moment – M (kN.m)
1	0.79	0.79	0.396
2	5.22	5.22	2.610
3	5.73	5.73	2.865
4	9.77	9.77	19.253

As discussed in section 2.1.4 of the literature review, sandwich panels have many different failure modes which means that it is important to consider all potential failures modes simultaneously, as failure that occurs at the lowest load will occur first and therefore be the dominant, contributing factor in the flexural strength. Due to this, the equivalent strength for each failure mode must be found and compared against all of the other potential failures, and the lowest load at which any particular failure will occur is adopted.

The literature review uncovered some analytical methods for analysing low strength core materials (such as the foam materials typically used), although Manalo and Aravinthan (2012) present a successful approach to describing the flexural behaviour of a high strength core material used in a sandwich beam structures, which showed a good agreeance with experimental data. The method of Fibre Model Analysis (FMA) in a simplified form was used to analyse the section in a similar manner to that of a typical concrete beam, and will be used in this report to analyse the test data outlined in section 4 of this report.

The main failure modes that will be considered in this analysis is centred around the shear and bending failures of both the skin materials and core where applicable which will allow a direct comparison to what was observed during testing. The possible failure modes for consideration are given as:

- Shear Failure (Core)
- Bending Failure (Skin)
- Bending Failure (Core)

5.2.2.1 Shear Failure (Core)

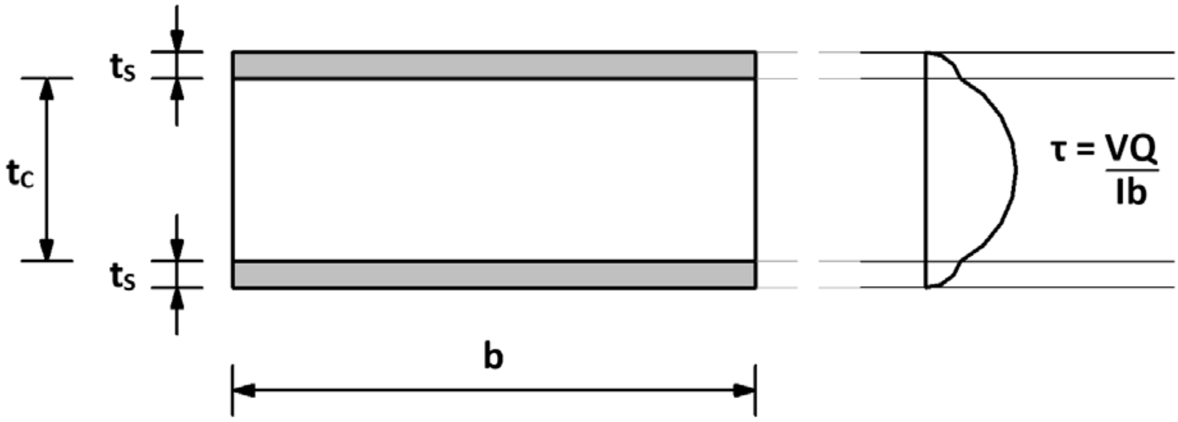

Figure 5.5 - Shear Failure of the Core

The shear distribution diagram through the section of a typical sandwich member can be seen in Figure 5.5, with the factors that contribute to the core shear capacity (τ_c) outlined in the following expression:

$$\tau_c = \frac{VQ_A}{Ib}$$

Where:

$$V = \text{Shear Force} - \text{Maximum} \;\; (\text{kN})$$

$$\text{Varies with loading/support arrangment} - \text{Refer to section 5.2.2}$$

$$Q_A = \text{Statical moment of area } (\text{m}^3) \text{ Neglecting the contribution of the skin material}$$

$$Q_A = \frac{bt_c}{2} \times \frac{t_c}{4} = \frac{bt_c^2}{8}$$

$$I = \frac{bt_c^3}{12}$$

$$b = \text{Width of panel}$$

Which when combined, gives the simplified expression:

$$\boldsymbol{\tau_c = \frac{3V}{2t_cb}}$$

5.2.2.2 Bending Failure (Skin)

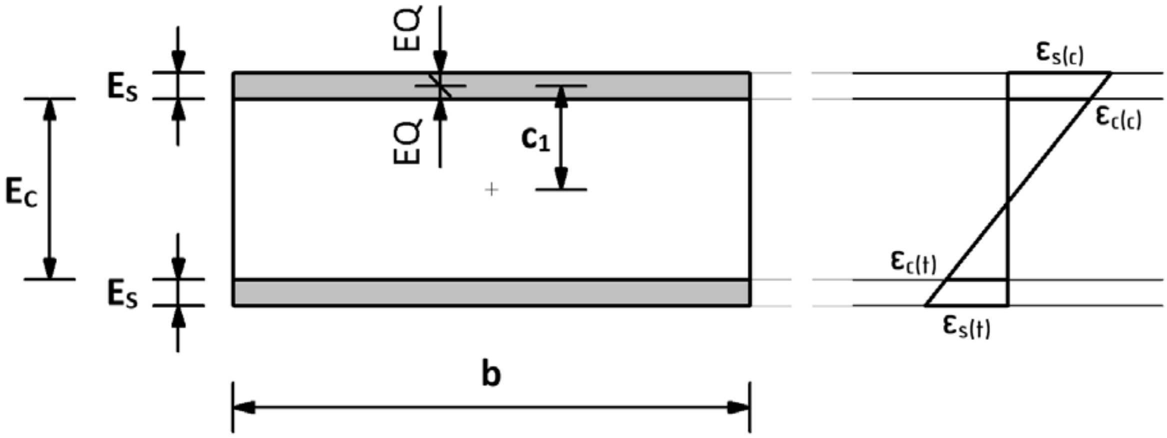

Figure 5.6 - Bending Stress in the Skin

The strain distribution through the section of a typical sandwich member can be seen in Figure 5.6, with the factors that contribute to the bending stress capacity of the skin material (σ_s) outlined in the following expression for both tension (t) and compression (c):

$$\boldsymbol{\sigma}_{\mathbf{s(t)}} = \frac{\mathbf{Mc_1}}{\mathbf{EI_{Total}}}\mathbf{E_s}$$

$$\boldsymbol{\sigma}_{\mathbf{s(c)}} = \frac{\mathbf{Mc_1}}{\mathbf{EI_{Total}}}\mathbf{E_s}$$

Where:

$\mathrm{M} = \text{Bending moment induced by applied load} - \text{Maximum} \;\; (\mathrm{kN.m})$

$\text{Varies with loading/support arrangment} - \text{Refer to section 5.2.2}$

$\mathrm{c_1} = \text{Distance of extreme fibers from centroid}$

$$= \frac{\text{Core Thickness } (\mathrm{t_c})}{2} + \frac{\text{Skin Thickness } (\mathrm{t_s})}{2}$$

$\mathrm{EI_{Total}} = \text{Total Flexural Rigidity of the section } (\mathrm{kN.m^2})$

$\mathrm{E_s} = \text{Modulus of elasticity of the skin material (kPa)}$

5.2.2.3 Bending Failure (Core)

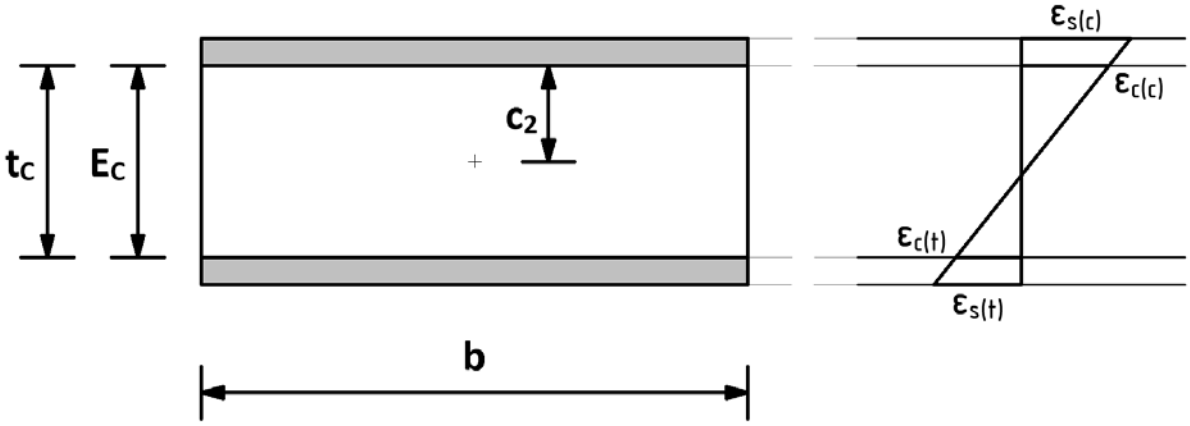

Figure 5.7 - Bending Stress in the Core

The strain distribution through the section of a typical sandwich member can be seen in Figure 5.7, with the factors that contribute to the bending stress capacity of the core material (σ_c) outlined in the following expression:

$$\boldsymbol{\sigma_c = \frac{Mc_2}{EI_c} E_c}$$

Where:

$\mathrm{M} = \text{Bending moment induced by applied load} - \text{Maximum} \;\; (\mathrm{kN.m})$

$\text{Varies with loading/support arrangment} - \text{Refer to section 5.2.2}$

$c_2 = \text{Half of the core thickness}$

$= \frac{\text{Core Thickness } (t_c)}{2}$

$EI_c = \text{Flexural Rigidity of the core } (\mathrm{kN.m^2})$

$E_c = \text{Modulus of elasticity of the core material } (\mathrm{kPa})$

5.2.2.4 Test Sample 1

Although test sample 1 has no skin materials present, it remains a test that can be used for validation of the analysis method. The analysis for test sample 1 will therefore only need to consider the core material, whereas the analysis for test samples 3 and 4 will consider the effect of the skin materials as well.

The shear strength is not specified by Hebel (the manufacturer of the AAC core panel), and so an estimation has been adopted for the shear strength as 1/8th of the compressive strength (Mathey 1988). Hebel provide the compressive strength and flexural characteristics and were adopted in the analysis of each test sample. The results for the analysis and a comparison to the test failures for test sample 1 can be seen in Table 5.3 below:

Table 5.3 - Test Sample 1 Analysis

Failure	Calculations	Theoretical Failure	Actual Failure	Did Test Failure Via This Mode	Should It Have Failed
Shear (Core)	$\tau_c = \frac{3V}{2t_c b}$ (Formula) Where $\tau \approx \frac{2.8 \times 10^3 \text{kPa}}{8}$ $350 = \frac{3 \times V}{2 \times 0.075 \times 0.6}$	10.5 kN	0.79 kN	No	No
Bending (Core)	$\sigma_c = \frac{Mc_2}{EI_c} E_c$ (Formula) Where $\sigma_c \approx 0.54 \times 10^3 \text{kPa}$ $540 = \frac{M \times 0.0375}{33.6} \times 1595 \times 10^3$	0.3 kN.m	0.395 kN.m	Yes	Yes

5.2.2.1 Test Sample 3

Test sample 3 has similar characteristics to test sample 1, with the addition of the steel skins which incur additional analysis and checks, refer to Table 5.4.

Table 5.4 - Test Sample 3 Analysis

Failure	Calculations	Theoretical Failure	Actual Failure	Did Test Failure Via This Mode	Should It Have Failed
Shear (Core)	$\tau_c = \frac{3V}{2t_c b}$ (Formula) Where $\tau \approx \frac{2.8 \times 10^3 \mathrm{kPa}}{8}$ $350 = \frac{3 \times V}{2 \times 0.075 \times 0.6}$	$10.5\ \mathrm{kN}$	$5.73\ \mathrm{kN}$	Yes	No
Bending (Core)	$\sigma_c = \frac{Mc_2}{EI_c} E_c$ (Formula) Where $\sigma_c \approx 0.54 \times 10^3 \mathrm{kPa}$ $540 = \frac{M \times 0.0375}{33.6} \times 1595 \times 10^3$	$0.3\ \mathrm{kN.m}$	$2.87\ \mathrm{kN.m}$	No	Yes
Bending (Skin)	$\sigma_{s(t)} = \sigma_{s(c)}$ for steel skins $\sigma_s = \frac{Mc_1}{EI_{Total}} E_s$ (Formula) Where $\sigma_s \approx 300 \times 10^3 \mathrm{kPa}$ $300 \times 10^3 = \frac{M \times 0.037775}{222} \times 200 \times 10^6$	$8.8\ \mathrm{kN.m}$	$2.87\ \mathrm{kN.m}$	No	No

5.2.2.1 Test Sample 4

Test sample 4 with its thicker core material is analysed in a similar manner to that of test sample 3 with the geometric adjustments to suit as shown in Table 5.5.

Table 5.5 - Test Sample 4 Analysis

Failure	Calculations	Theoretical Failure	Actual Failure	Did Test Failure Via This Mode	Should It Have Failed
Shear (Core)	$\tau_c = \frac{3V}{2t_c b}$ (Formula) Where $\tau_c \approx \frac{2.8 \times 10^3 \text{kPa}}{8}$ $350 = \frac{3 \times V}{2 \times 0.15 \times 0.6}$	21 kN	9.77 kN	No	No
Bending (Core)	$\sigma_c = \frac{Mc_2}{EI_c} E_c$ (Formula) Where $\sigma_c \approx 0.54 \times 10^3 \text{kPa}$ $540 = \frac{M \times 0.075}{452.4} \times 1595 \times 10^3$	2.04 kN.m	19.25 kN.m	No	Yes
Bending (Skin)	$\sigma_{s(t)} = \sigma_{s(c)}$ for steel skins $\sigma_s = \frac{Mc_1}{EI_{Total}} E_s$ (Formula) Where $\sigma_s \approx 300 \times 10^3 \text{kPa}$ $300 \times 10^3 =$ $\frac{M \times 0.075275}{1200} \times 200 \times 10^6$	23.9 kN.m	19.25 kN.m	No	No

5.2.2.2 Discussion

It can be seen that for test sample 1, it was calculated that the failure would occur as a result of a bending failure not shear. When loaded during the test, the sample failed in the predicted manner, though this bending failure load (0.3 kN.m) was slightly lower than what the test actually achieved (0.395 kN.m). Though the presence of the cracking that occurred prior to the eventual complete failure goes some way to explaining the slight discrepancy between the two. The prediction is somewhat a conservative estimate based on the test data and it would be ideal to replicate this test multiple times to collate more data to compare against. Therefore, it can generally be stated that the sample where no skin materials were present was replicated with relative accuracy via the analytical methods presented in this chapter.

Test sample 3 highlighted some differences between the expected and actual failures, with the shear failure of the core occurring earlier than theoretically expected. This highlights some of the lack of accurate and consistent literature for predicting shear strengths for the AAC material. It can also be seen that the bending force that the core was subject to suggests that it should have failed during the test (at the same applied load as test sample 1), though the test clearly shows that as the core is bonded to the skin material, any flexural cracking that may occur during loading is prevented from being widened as the bottom steel skin provides the tensile capacity of the structure. Lastly, the analysis suggested that the skin material would not fail under the applied load during test sample 3, which proved to be a correct statement, although the eventual test failure was unable to be determined as the core failed prior to this point. It was also observed and noted that skin debonding occurred during this test and this had some impact on the results, although this was more evident during the deflection comparisons as the shear strength of the core is independent to the skin bonding.

It was also seen that during the final test (sample 4), the specimen did not experience a particular significant failure, rather sheet wrinkling as a result of the concentrated loading configuration of a four-point bending test. The analysis predicts that the core would not fail from the shear force until 21kN of shear force was experienced, though the test did not reach this point to validate this. Similarly, the bending failure of the skin was estimated to occur as 23.9kN.m and the test was ceased at 19.25kN.m, which although close to the expected failure, it could not be confirmed. Similar to what was found in test sample 3, the bending failure of the core was not evident during the test despite the analysis suggesting its limited capacity, again suggesting that the skin material carries the bending forces, whilst the core material provides the shear resistance.

5.2.2.1 Outcomes

As a result of the findings, it is clear that the bending failure of the AAC core material is not significant in the analysis of the system as the skin materials carry the load when bonding between the two materials is evident. Therefore, during the predictive analysis in section 7 of this report, this particular mode of failure will not need to be considered.

It can also be seen that the shear capacity of the AAC core is potentially less than expected based on the literature found. In lieu of finding additional accurate information regarding the shear strength, it is reasonable to scale the shear strength back from what the test results provided as shown below:

$$\tau_c = \frac{3V}{2t_c b} \text{ Where V is Shear Value At Test Failure (5.73 kN for test sample 3)}$$

$$\tau_c = \frac{3\times 5.73}{2\times 0.075\times 0.6} = 191\text{kPa}$$

$$\frac{\text{Shear Strength}}{\text{Compressive Strength}} = \frac{191}{2850} \approx \text{Shear Srength 7\% of compressive strength}$$

This will be adopted in section 7 of this report and recommendations made for this area of failure will be given when considering the potential future work of this system.

The bending failure of the skin material was not distinctively evident during either the testing or analysis of the system, although consideration still needs to be made for this type of failure. It is noted that the point at which the steel skin material fails in bending is likely to occur after the core has failed via shear or the serviceability deflection criteria has been exceeded.

6 Potential Applications

6.1 Introduction

In order to determine the areas in which the AAC core, steel stressed-skin combination can be used as part of a building structure, a number of potential options will be explored:

- Wall
- Roof
- Floor
- Permanent Formwork

To ensure the variability between each option is achieved, with the aim of being able to comment on the characteristics of each application and further recommendations, some basic criteria will be addressed including requirements from:

- 2019 National Construction Code's (NCC) Building Code of Australia (BCA) Volume 1
- AS/NZS 1170.0 (2002) – Structural Design Actions: General principles
- AS/NZS 1170.1 (2002) – Structural Design Actions: Permanent, imposed and other actions
- AS/NZS 1170.2 (2011) – Structural Design Actions: Wind Actions
- AS 3610 (1995) – Formwork for Concrete
- AS 4055 (2012) – Wind Loads for Housing

The criteria will give consideration for deemed-to-satisfy provisions for basic fire resistance requirements, noting that there are many other factors that are potentially in a building system that must be considered to ensure all of the fire rating necessities are met and compliance is achieved. The literature review highlighted that significant evaluation, testing and simulation data would be required for this new AAC and steel composite structure to prove that it can comply with the fire resistance levels. The purpose of this report is to form a wholistic understanding of the potential applications that the concept may possess and in many real-world cases each particular building application has to be judged by a suitably qualified certifier and engineer on its own individual features and make-up, some of which may not be considered in this report.

Therefore, this dissertation will not specify minimum FRL criteria for each application as there is a large degree of variation that can occur which can depend on a myriad of factors. As a reference to potential FRL's, some of the relevant information from the National Construction Code has been included in the appendices. Specifically, Appendix D has information regarding the factors that outline the class of a building (Class 1 – 10), which then gets carried through to Appendix E to choose an appropriate type of construction (Type A – C) and finally Appendix F has the actual FRL's for Type A construction (as a worst-case scenario). The National Construction Code has the remaining information for the cases not given in the appendices.

The serviceability and ultimate limit states given in the Australian Standards outline the procedures, criteria and load combinations for consideration, and this section of the report is centred around understanding the expected loading conditions that each building component would be subjected to for each respective application. The information given in each of the relevant standards will be used to establish and define the criteria which will then be used as the basis of the predictive analysis in this report.

Some consistent assumptions carried through this analysis will be that the proposed panel is to be used in building applications that are consistent with:

- Importance level of 3 (as stipulated by the BCA Volume 1)
- Building class of 5/6 (as stipulated by the BCA Volume 1, refer to Table D.1 in Appendix D)
- Type A Construction (as stipulated by the BCA Volume 1, refer to Table E.1 in Appendix E)

Another basic assumption is that the system will not be used in a cyclonic region in Australia. This will simplify some of the requirements that are associated with cyclonic regions such as the complex requirements of low cycle fatigue testing for cladding elements such as the proposed product being considered here. This doesn't render the concept unusable for cyclonic regions of Australia, though it simplifies some of the requirements when determining the basic suitability for the proof of concept.

6.2 Serviceability Limit State

AS/NZS 1170.0 (2002) – Structural Design Actions: General principles provides the serviceability limit states to be used in checking the serviceability of a structural element. The point at which the item in question exceeds the limits on deformation, vibratory response, degradation or other physical aspects is considered to be the conditions for serviceability failure, and therefore the conditions that the structure experiences must be less than the limiting serviceability condition. This is shown in the below expression:

$$\delta \geq \delta_l$$

Where:

$\delta =$ Value of the serviceability parameter determined on the basis of design actions (Clause 4.3)

$\delta_l =$ Limiting value of the serviceability parameter

A number of combinations are presented in Clause 4.3 of AS1170.0 and must be considered (as shown in Table 6.1), ensuring that all potential loading cases are considered, and the most extreme loading condition is used to check the capabilities of the structure against.

Table 6.1 - Serviceability Ultimate Limit State Combinations

Limiting Value Parameter (δ_l) Reference Number	Formula	Description
a	G	Permanent action only
b	$\psi_s Q$	Short-term and imposed action
c	$\psi_l Q$	Long-term imposed action
d	W_s	Wind action
e	E_{serv}	Earthquake action
f	Serviceability values of other actions	As appropriate

Table C1 in AS1170.0 provides guidance on serviceability limits for the design of members and specifically deflection limits, relating to an annual probably of exceedance if 1/25. The criteria in this table will be used in this report (where possible) as the serviceability requirements for the potential applications of the sandwich panel and can be seen in Table G.1 in Appendix G. In a similar manner to the ultimate limit state scope, the serviceability limit states will only take the relevant design actions into consideration, refer to section 6.3 of this report for the details of the exclusions.

6.3 Ultimate Limit State

AS/NZS 1170.0 (2002) – Structural Design Actions: General principles provides the ultimate limit state requirements to be used when checking the strength of a structural element. The point at which the item in question reaches a point of collapse, rupture or excessive deformation it is considered to have reached the conditions for failure in strength, and therefore the design capacity must be equal to or greater than the design action effect, as shown in the below expression:

$$R_d \geq E_d$$

Where:

$R_d = \text{Design capacity (equal to } \emptyset R)$

$E_d = \text{Design action effect}$

A number of combinations are presented in Clause 4.2.2 of AS1170.0 and must be measured, ensuring that all potential loading cases are considered (as shown in Table 6.2), and the most extreme loading condition is used to check the capacity of the structure against.

Table 6.2 - Strength Ultimate Limit State Combinations

Design Action Effect (E_d) Reference Number	Formula	Description
a	$1.35G$	Permanent action only
b	$1.2G + 1.5Q$	Permanent and imposed action
c	$1.2G + 1.5\psi_l Q$	Permanent and long-term imposed action
d	$1.2G + W_u + \psi_c Q$	Permanent, wind and imposed action
e	$0.9G + W_u$	Permanent and wind action reversal
f	$G + E_u + \psi_E Q$	Permanent, earthquake and imposed action
g	$1.2G + S_u + \psi_c Q$	Permanent action, actions given in Clause 4.2.3 AS1170.0 and imposed action

In this report, the design actions are related to structures located outside the cyclonic regions of Queensland, and therefore a number of actions will be omitted as their characteristics are not cause for consideration such as snow and earthquake loading conditions. The factors relating to these items are not as significant as the remaining design actions, which will be the prevalent items that create the worst-case scenario that is required to be adopted.

6.4 Wall

The application for an AAC core, steel skinned sandwich panel as a wall system could have a multitude of variations and be presented as a solution for many different walling components within a building. To further narrow the scope in this area, the focus of this section of the report will be a panel used as a replacement for traditional girt framing. Typically, this would necessitate light steel framing which spans between largely spaced steel columns which form the main part of a portal frame segment. In that instance the steel sheeting is then fixed to the girt framing and any insulation required to be installed on the inside of the framing. The proposed system is intended to be installed horizontally, spanning to the main portal frame column which can be seen in Figure 6.1, in lieu of the multiple part system currently being used throughout the construction industry.

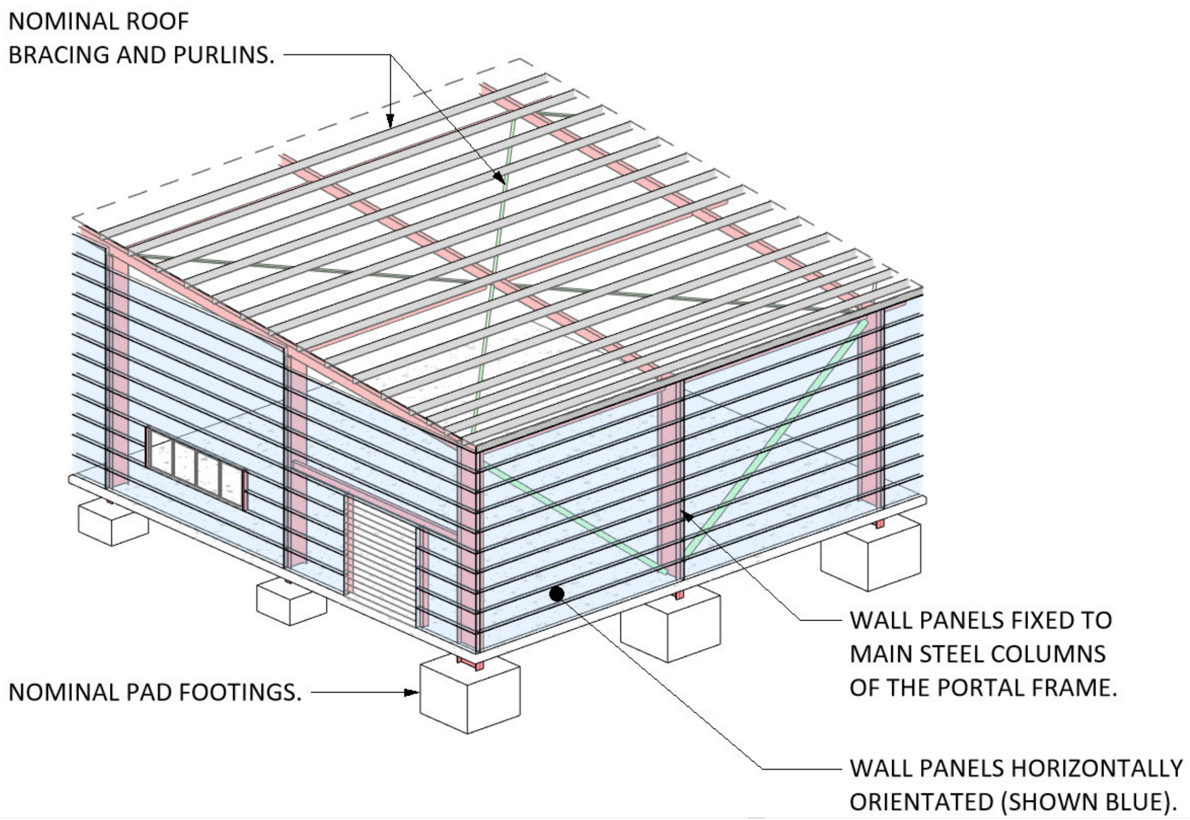

Figure 6.1 - Wall Application

6.4.1 Serviceability Limit State

The suggested serviceability limit state criteria given in AS1170.0 (refer to Table G.1 in Appendix G) for a general wall which is face loaded is given as Height/150 for the mid-height deflection due to wind loading (W_s). The objective of this criteria is to prevent discerning movement by the wall, although this condition will be replaced with Height/250 in this report to cover the more conservative requirements for preventing façade damage for walling systems given in AS1170.0.

The service wind action W_s is difficult to define as each potential site where the system may be used is different, which means that the site wind speed and subsequent pressures change for every building. The local site wind speed changes with factors such as the region in which it's located in, the terrain category, the topography and shielding factors.

Therefore, for the purpose of this report, the Code AS 4055 (2012) – Wind Loads for Housing will be used to get a generic serviceability wind actions for three different typical wind classification zones, being N1, N3 and N5 which correspond with 26, 32 and 47 m/s respectively for the design gust wind speeds. Noting that although this is not a blanket ruling for all potential wall uses, it will provide a reasonable guide as to how the system is likely to perform in differing local environments. The code gives net pressure coefficients, which form the basis of serviceability limit state design pressures given in Table H.1 in Appendix H, and this has provided the quantifiable W_s values to be used in this report. The pressures along any given wall will vary, though for the purpose of this analysis this report will use a worst-case pressure that occurs within 1200mm corners and can be seen to be -0.55, -0.83 and -1.79kPa for the N1, N3 and N5 wind classifications respectively.

Another serviceability requirement stipulated in AS1170.0 is the ability to resist an imposed point load of 0.7kN, such as a running person falling against a wall, and this must be within a Height/200 limit, and also less than 12mm.

6.4.2 Ultimate Limit State

When considering the ultimate limit state combinations from a wall system, the most significant loading is the horizontal wind action W_u, and less consideration is required from the permanent self-weight of the panel in the vertical direction. As a result, the expected ultimate strength pressures must be determined in order to check the capabilities of the panel against the expected loading conditions.

In a similar manner to the serviceability considerations for wall pressures, AS4055 also provides pressures for varying wind classifications, which can be seen in Table I.1 given in Appendix I. Once again, to assess the capabilities of the wall system in different local environments the three wind classifications of N1, N3 and N5 will be used, which return maximum pressures within 1200mm of corners of -0.94, -2.03 and -4.44kPa respectively.

6.5 Roof

In a roofing application, the sandwich panel has the potential to be used in a similar manner to the wall panelling to a large steel portal framed structure, down to a much smaller structure such as a carport or patio roof. The concept has the potential to apply to almost any roof structure with suitable supports or as per the arrangement mentioned above, an example of which is shown in Figure 6.2. This system presents a simple alternative to current systems such as light steel purlins with roof sheeting or in lieu of timber rafters with battens supporting roof sheeting. This provides similar benefits to the walling application, with a single component having the potential to address the purpose of the many different materials used in current common construction techniques such as roofing systems.

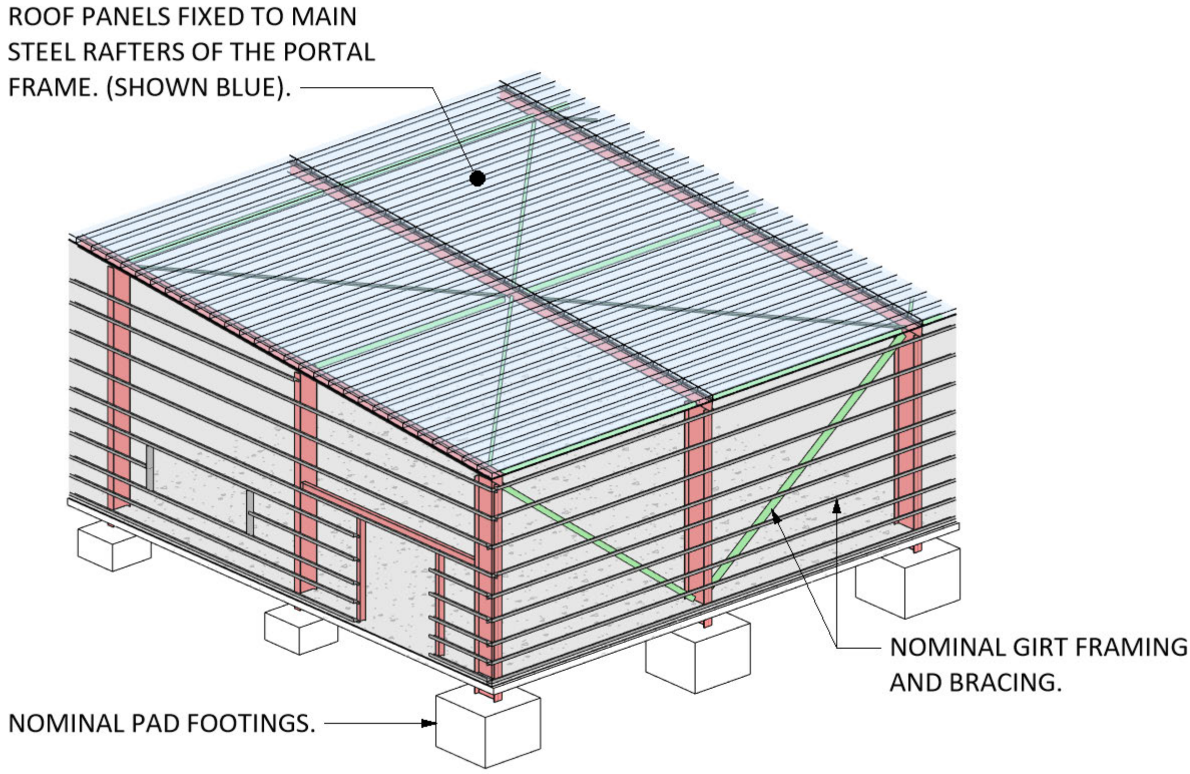

Figure 6.2 - Roof Application

6.5.1 Serviceability Limit State

For roof elements, AS1170.0 (refer to Table G.1 in Appendix G) has provisions for both roof cladding and roof-supporting applications. As the proposed system is a combination of a roof member as well as the cladding, it will need to satisfy all of the requirements listed. The roof cladding gives a de-coupling mid-span deflection limit of Span/120 for dead loads G and short-term imposed loads $\psi_s Q$.

Another criterion for roof members is set for controlling the mid-span sagging deflection as Span/300 for dead loads G and long-term imposed loads $\psi_l Q$. The permanent action varies with the span, core thickness, core density and sheet thickness and therefore is calculated for each specific arrangement, and the short and long-term factors for roof elements is given in AS1170.0 as 0.7 and 0.0 respectively for distributed imposed actions and 1.0 and 0.0 for concentrated imposed actions. The imposed actions for roof elements are given in AS1170.1 and for structural elements, both a uniformly distributed load and a concentrated load is required to be assessed. The uniformly distributed load is found by using the surface area of roof supported in the equation below and the concentrated load is given as 1.4kN.

$$\mathrm{Q} = \frac{1.8}{\mathrm{Area}} + 0.12 \;\geq 0.25\ \mathrm{kPa}$$

A similar Span/300 will be checked for the serviceability wind action, with AS4055 giving maximum inward pressures for N1, N3 and N5 wind classifications of +0.26, +0.39 and +0.84kPa and maximum outward pressures at corners within 1200mm of both edges of -1.06, -1.60 and -3.46kPa.

The roof cladding must also resist residual deformation when subject to an imposed Q load of 1 kN, a phenomenon which will not be a restrictive parameter with the AAC core directly beneath the cladding, in this case the steel skin material.

6.5.2 Ultimate Limit State

For the roof application, a number of different design action effects are required to be considered simultaneously, with the permanent action changing with dimensions, along with the imposed action varying with surface area in a similar manner to the details given for serviceability.

The wind action for both inward and outward wind directions also need to be considered, with AS4055 giving maximum inward pressures for N1, N3 and N5 wind classifications of +0.44, +0.95 and +2.07kPa and maximum outward pressures at corners within 1200mm of both edges of -1.81, -3.92 and -8.58kPa. The outward pressures are seen to be significant in comparison to the other actions present in this case and will likely dominate the calculated criteria.

6.6 Floor

The proposed building concept can be extended and applied to using it in a flooring system, which has the potential to be successfully implemented into general areas in domestic applications, mezzanine floors or even suspended floors in an office environment. This area of focus would aim to eliminate the need for basic timber floor framing in a residential application or even labour intensive conventional concrete suspended slabs formed on site. A nominal arrangement for such an application can be seen in Figure 6.3 below, with flexible options for supporting elements such as walls or general steel framing.

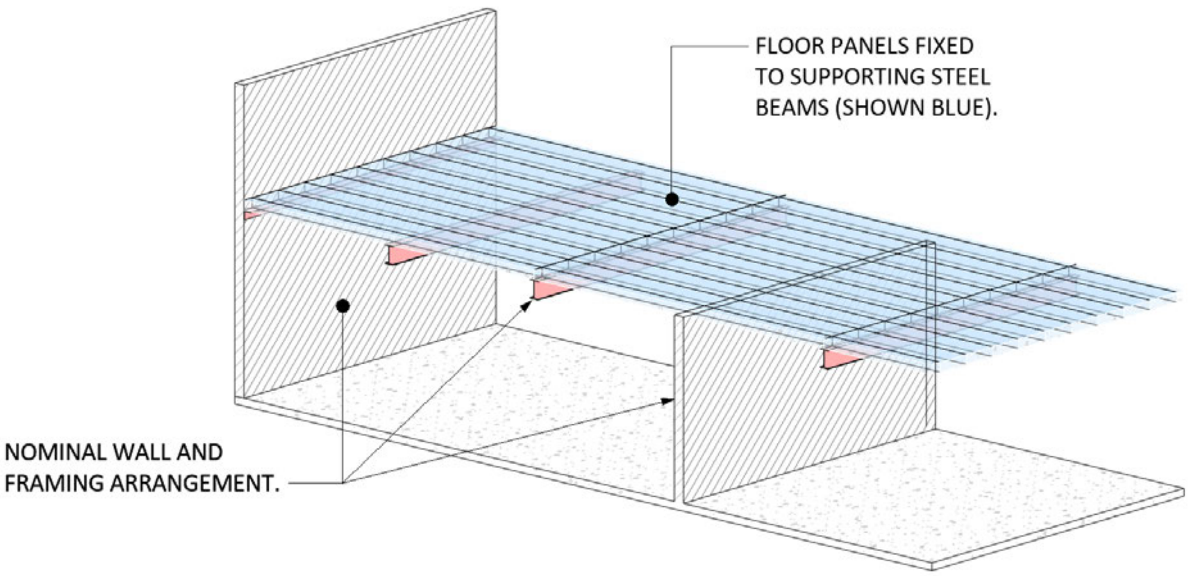

Figure 6.3 - Floor Application

6.6.1 Serviceability Limit State

Using Table G.1 in Appendix G which relates to AS1170.0, the serviceability requirements of interest for a floor cover two possible loading scenarios. The first is a basic vibration check for a imposed point load of 1.0kN, which requires the static midspan deflection to be less than 1 to 2mm as a guide noting that the code states that this is a general guide as to whether or not the floor system is likely to experience vibration concerns and that further investigation may be required case-by-case.

The second element response limits can be given as Span/400, and this is measured as mid-span deflection caused by a combination of both the dead load G and long-term imposed $\psi_l Q$. This deflection limit is set as to eliminate noticeable sag in the floor system, and is generally more stringent than the wall and roof applications outlined in the previous sections.

The permanent load G varies and is found in the same manner as discussed in section 6.5.1 of this report, and the long-term factor is given as 0.4 from AS1170.0 and the imposed uniformly distributed and concentrated action is 3.0kPa and 2.7kN respectively for the cover the applications outlined in section 6.6 of this report. Noting that there are many applications where the imposed load is much lower, for example in general residential applications the imposed action is given as 1.5kPa.

6.6.2 Ultimate Limit State

Considerations for floor ultimate limit states will be governed by the vertical permanent and imposed actions, and therefore the design action effect produced by 1.2G + 1.5Q will be the reference formula to use. As aforementioned, the permanent action will vary, and the imposed action is the same 3.0kPa and 2.7kN given for the serviceability limits state requirements.

6.7 Formwork

The option of using the AAC core and steel skins in combination to create a sandwich panel profile has the potential to be considered as a permanent formwork system for suspended slab systems. In combination with a topping slab, the concept has the potential to assist in the constructability of building systems by reducing costs and time associated with erecting and removing formwork. This can also have the flow on effect of having a thinner slab profile over the top in comparison to convention suspended slab systems, thus reducing the weight of each level in the building and having the potential to reduce associated structural member sizes as a result such as footings and supporting beams.

In terms of the applied loading, the system has to consider supporting the initial construction loads experienced prior to and including the pour of the topping slab. AS3610 (1995) Formwork for Concrete, provides the information in relation to the loading requirements that a formwork system must adhere to. The code itemises the construction cycle into three stages, which come into effect when considering the most adverse loading situation:

- Stage 1 – Prior to placement of concrete (including the handling and erection of the formwork and once the formwork structure is erected).
- Stage 2 – During the placement of concrete.
- Stage 3 – After placement of concrete (until the concrete is able to support the applied loads).

The code provides information for all types of loads that may be carried by formwork, though for this application only the vertical loads commonly experienced for formwork used for supporting suspended floors will be considered. This includes:

- The dead load (G), which is based on the self-weight of the panel – limited to the skin and core materials.
- The concrete load (G_c) is used to address the loading caused by the topping slab, which is 75mm in thickness which equates to approximately 1.8kPa.
- The live load (Q_{uv}, Q_c):
  - Stage 1 Q_{uv} – 1.0kPa
  - Stage 2 Q_{uv} – 1.0kPa
  - Stage 3 Q_{uv} – 1.0kPa
- The load from stacked materials (M)
  - Stage 1 M_1 – 4.0kPa
  - Stage 2 M_2 – 0.0kPa

- Stage 3 M_3 – 4.0kPa – the code stipulates that this load can be reduced if the loads from stacked materials are nominated in the relevant documentation and adequate control is achieved on site. For the purpose of this report, it is assumed that in stage 3, no additional loading from stacked materials is required as with common practice with similar products as no stacked materials are allowed until the slab has reached a nominal 15MPa strength.

This report excludes the following load considerations presented in AS3610, due to either being irrelevant or require consideration of many job specific variables to be known:

- Multistorey loading
- Water loading
- Miscellaneous loading
- Lateral concrete pressure
- Wind loading
- Horizontal impact

As the system is intended to be permanent, it also needs to satisfy the general floor criteria, as detailed in section 6.6, in relation to both serviceability and strength conditions as part of the full composite section, including the topping slab.

6.7.1 Serviceability Limit State

As mentioned above, the serviceability criteria must take into account both the formwork and permanent flooring requirements.

The loading conditions that the supporting structure must support for a typical formwork system needs to include all load combinations given in Table 4.5.1 in AS3610 (refer to Table J.1 in Appendix J) and a check for stiffness to be completed against equations 11-13. Being placed horizontally in orientation requires the self-weight of each panel to be considered in addition to the uniform load given. Typically, the most significant load combination for the deflection to be checked against is equation number 13 from the table, and is given below:

$$\text{Equation 13 From Table 4.5.1 in AS3610} = (\mathrm{G} + \mathrm{G_c} + \mathrm{M_3}) \times 1.0$$

The allowable deflection for the formwork component is set as Span/250 which is a general guide given in AS3600 (2018) – Concrete Structures.

For the flooring considerations (once the topping slab is poured and strength is achieved), the same criteria outlined in section 6.6.1 of this report will be used, with considerations made for the inclusion of the topping slab. The flexural rigidity (EI) value will change as the neutral axis is no longer central to the panel as it has moved toward upward due to the influence of the topping slab (an example calculation for the neutral axis location can be seen in Appendix C).

6.7.2 Ultimate Limit State

In a similar approach for the serviceability requirements, AS3610 provides loading combinations for the ultimate strength that a formwork system must resist. Once again, Table 4.5.1 from AS3610 (refer to Table J.1 in Appendix J) has the equations listed 1-10, and equation 1 typically provide the largest loading requirements for the system being considered here:

$$\text{Equation 1 From Table 4.5.1 in AS3610} = (1.25\text{G} + 1.5\text{Q}_{\text{uv}} + 1.5\text{M}_1) \times 1.3$$

Once the topping slab is poured and strength is achieved, the same criteria outlined in section 6.6.2 of this report needs to be met, with considerations made for the additional loading from the topping slab. With the introduction of the topping slab, it becomes unlikely that shearing of the core will occur and will therefore be omitted for the flooring component of the strength calculations, due to the topping slab providing the shear resistance. The tensile bending capacity of the structure remains the bottom skin, though the top compressive strength is provided by the topping slab, and will require checking in lieu of the top skin material.

6.8 Summary

The previous sections of this report provide an in-depth review of the conditions that a typical steel skinned, AAC wall, roof, floor and formwork panel would be likely to be subject to from a design sense. Table 6.3 and Table 6.4 provide an overall summary of the criteria being considered for each structural use, showing how the serviceability and ultimate requirements differ between each application. This provides the analysis and the input information for the decision-making process charts provided in section 7.4.1 of this report. The variation in criteria once analysed will deliver the data required to make relevant comments and recommendations for the suitability of each different potential area of use.

Table 6.3 - Summary of Serviceability Requirements

Structural Use	Serviceability Requirements				
	Phenomenon Controlled	**Restriction**	**Applied Action**	**Applied Force**	**Element Response**
Wall	Impact	Mid-Span Deflection	Q	0.7kN	Span/200 but <12mm
	Discerned Movement / Façade Damage	Mid-Span Deflection	W_s	N1 – -0.55kPa N3 – -0.83kPa N5 – -1.79kPa	Span/250
Roof	De-Coupling	Mid-Span Deflection	$G + \psi_s Q$	G Varies $\psi_s = 0.7$ Q Varies	Span/120
	Sag	Mid-Span Deflection	$G + \psi_l Q$	G Varies $\psi_l = 0$ Q Varies	Span/300
	Sag	Mid-Span Deflection	W_s	N1 – -1.06kPa N3 – -1.60kPa N5 – -3.46kPa	Span/300
Floor	Vibration	Static Mid-Span Deflection	Q	1.0 kN	< 1-2mm
	Noticeable Sag	Mid-Span Deflection	$G + \psi_l Q$	G Varies $\psi_l = 0.4$ Q = 3.0kPa & Q = 2.7kN	Span/400
Formwork	Sag For Construction Formwork Loading	Mid-Span Deflection	Varies	$G + G_c + M_3$	Span/250
	Conditions for vibration and noticeable sag as per floor requirements (see above)				

Table 6.4 - Summary of Ultimate Requirements

<table>
<tr><th rowspan="2">Structural Use</th><th colspan="3">Ultimate Strength Requirements</th></tr>
<tr><th colspan="2">Applied Action</th><th>Applied Force</th></tr>
<tr><td>Wall</td><td colspan="2">W_u</td><td>N1 – 0.94kPa
N3 – 2.03kPa
N5 – 4.44kPa</td></tr>
<tr><td rowspan="3">Roof</td><td rowspan="3">Maximum</td><td>1.2G + 1.5Q</td><td rowspan="3">G Varies
Q Varies
W_u N1 – +0.44, -1.81kPa
W_u N3 – +0.95, -3.92kPa
W_u N5 – +2.07, -8.58kPa
$\psi_c = 0.0$</td></tr>
<tr><td>$1.2G + W_u + \psi_c Q$</td></tr>
<tr><td>$0.9G + W_u$</td></tr>
<tr><td>Floor</td><td colspan="2">1.2G + 1.5Q</td><td>G Varies
Q = 3.0kPa</td></tr>
<tr><td rowspan="2">Formwork</td><td colspan="2">Varies</td><td>$(1.25G + 1.5Q_{uv} + 1.5M_1) \times 1.3$</td></tr>
<tr><td colspan="3">Conditions as per floor requirements (see above)</td></tr>
</table>

6.9 Variables

In order develop further understanding and retrieve data as to how each of the factors that contribute to the makeup of the sandwich panel effects each potential application, the following items will be varied:

- Core density (and the material properties that vary with the change in density)
- Core thickness
- Span
- Steel skin thickness

6.9.1 Core Density

The research completed in section 2.3.3 of this report uncovered that the variation in density of AAC has a large bearing on the properties. Hebel have four available densities available for production with the properties given in Table 6.5 below. This data is consistent with the expectations found during the literature review and will therefore be used for the predictive analysis in section 7 to best relate the analysis to the commercially available AAC material in the Australian market.

Table 6.5 - AAC Properties With Varying Density

Dry Density (kg/m^3)	Modulus of Elasticity E (MPa)	Compressive Strength f' (MPa)	Flexural Yield Strength σ (MPa)	Shear Yield Strength τ (MPa)
400	1275	> 2.4	> 0.45	0.24
510	1595	> 2.8	> 0.54	0.28
550	1755	> 4.0	> 0.72	0.4
580	1875	> 4.0	> 0.72	0.4

6.9.2 Core Thickness

A variety of core thicknesses will be covered, in 25mm increments starting with 75mm tailored towards the smaller spans and applications with less applied loading, and extending to 200mm to suit the supporting higher loads between more significant spans. This will allow for recommendations to be made and comment on the capabilities and limitations that particular core thicknesses have.

6.9.3 Span

One of the key factors in exploring the potential of the concept is its ability to span different distances to suit a number of different potential scenarios. Therefore, the analysis will focus on spans increasing at equal increments of 0.5 metres, starting at a small span of 1.5 metres and continuing up to 6.0 metres. The results found may indicate limitations in use or otherwise, to assist in making recommendations and suitability of the concept.

6.9.4 Steel skin thickness

The steel sheet options commercially available are vast, although to get an understanding of how the variation in thickness can influence the use of sandwich panel in conjunction with the AAC core, the following sheet thicknesses will be explored to provide enough variability to understand the influence that sheet thickness has on the overall performance of the system:

- 0.40mm
- 0.55mm
- 0.95mm
- 1.95mm

The strength of the steel sheets under consideration will be based upon a 300MPa strength guide and an approximate density of 8000kg/m^3 will be adopted when considering the permanent loading conditions.

7 Predictive Analytical Analysis

7.1 Introduction

The testing analysis undertaken in section 5.2 allows further manipulation and extrapolation to cover other potential loading applications and uses for the structural system. As the four-point bending test is a specific loading arrangement that is unlikely to be replicated in a real-world situation, a uniformly distributed pressure load will be used to connect the predictive analysis calculations to likely potential applications.

In a similar manner to the previous section, both serviceability and strength failures will need to be calculated to ensure the appropriate performance is achieved, with the approach of the structure acting as a holistic system, with all type of failures considered as having the potential to limit the load carrying ability of each panel combination.

7.2 Serviceability

Figure 7.1 shows the deflected shape that correlates with the static loading condition that a beam experiences during a uniformly distributed loading arrangement consistent with typical real-world relevant loading situations.

The relationship between the applied load, flexural rigidity, span and geometric arrangement give the following expression:

Deflection (maximum at center of span)$\Delta = \frac{5wL^4}{384EI}$

Where:

$\Delta = \text{Deflection (m)}$

$w = \text{Applied Load (kN/m)}$

$L = \text{Span (m)}$

$EI = \text{Flexural Rigidity (kN/m}^2\text{)}$

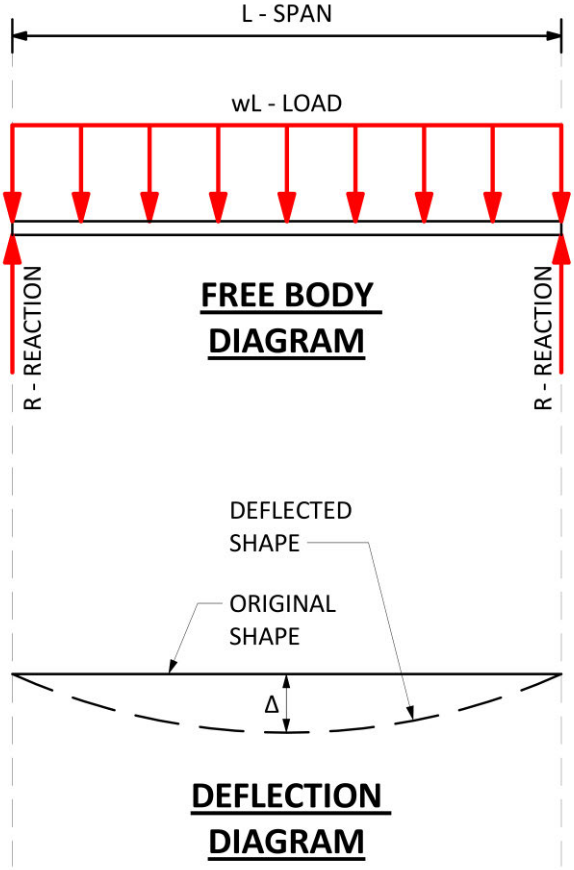

Figure 7.1 - Uniformly Distributed Load Deflection Diagram

The flexural rigidity can be found using the same approach given in section 5.2.1 (and discussed in section 6.7.1 for the formwork application with inclusion of the topping slab as shown in Appendix C), using the material and dimensional properties required for each arrangement.

The summary of flexural rigidity values that correspond with each core thickness, core density and skin thickness that will be investigated and utilised for the span table calculation can be seen in Table 7.1 (no topping slab present for wall, roof and floor applications) and Table 7.2 (where topping slab is present for formwork applications after stage 3 of the construction cycle). This data will be used when determining the serviceability deflection requirements and the bending capacity of each structure.

Table 7.1 - Flexural Rigidity (EI) Values for Combinations (No Topping Slab)

EI Values For Combinations – No Topping Slab (kN.m^2)					
Core Thickness	Core Density (kg/m^3)	Skin Thickness (mm)			
		0.40	0.55	0.95	1.95
75mm	400	163	215	356	720
	510	170	222	362	727
	550	173	225	366	730
	580	176	228	368	732
100mm	400	306	397	645	1280
	510	322	413	661	1296
	550	330	421	669	1304
	580	336	427	675	1310
125mm	400	502	645	1029	2010
	510	533	676	1060	2042
	550	549	692	1076	2057
	580	561	703	1087	2069
150mm	400	941	1146	1697	3100
	510	995	1200	1751	3154
	550	1022	1227	1778	3181
	580	1043	1248	1798	3201
175mm	400	1379	1658	2405	4304
	510	1465	1744	2491	4390
	550	1508	1786	2534	4433
	580	1540	1819	2566	4465
200mm	400	1917	2281	3255	5725
	510	2045	2409	3383	5853
	550	2109	2473	3447	5917
	580	2157	2521	3495	5965

Table 7.2 - Flexural Rigidity (EI) Values for Combinations (With Topping Slab)

EI Values For Combinations – With Topping Slab ($kN.m^2$)					
Core Thickness	Core Density (kg/m^3)	Skin Thickness (mm)			
		0.40	0.55	0.95	1.95
75mm	400	1533	1744	2277	3469
	510	1603	1811	2337	3516
	550	1637	1844	2367	3540
	580	1663	1868	2389	3557
100mm	400	2151	2450	3207	4917
	510	2276	2569	3315	5003
	550	2337	2628	3368	5045
	580	2383	2672	3408	5077
125mm	400	3060	3456	4468	6774
	510	3257	3646	4640	6913
	550	3353	3739	4725	6982
	580	3425	3808	4788	7033
150mm	400	4574	5075	6359	9315
	510	4859	5349	6609	9521
	550	4998	5483	6732	9623
	580	5100	5583	6823	9698
175mm	400	5920	6553	8182	11947
	510	6328	6947	8543	12246
	550	6526	7140	8720	12394
	580	6673	7282	8851	12503
200mm	400	7514	8296	10309	14978
	510	8075	8839	10808	15395
	550	8348	9103	11051	15599
	580	8549	9298	11231	15751

7.3 Strength

Figure 7.2 depicts the shear and bending moment diagrams that correlates with the static loading condition that a beam experiences during a uniformly distributed loading arrangement consistent with typical real-world relevant loading situations.

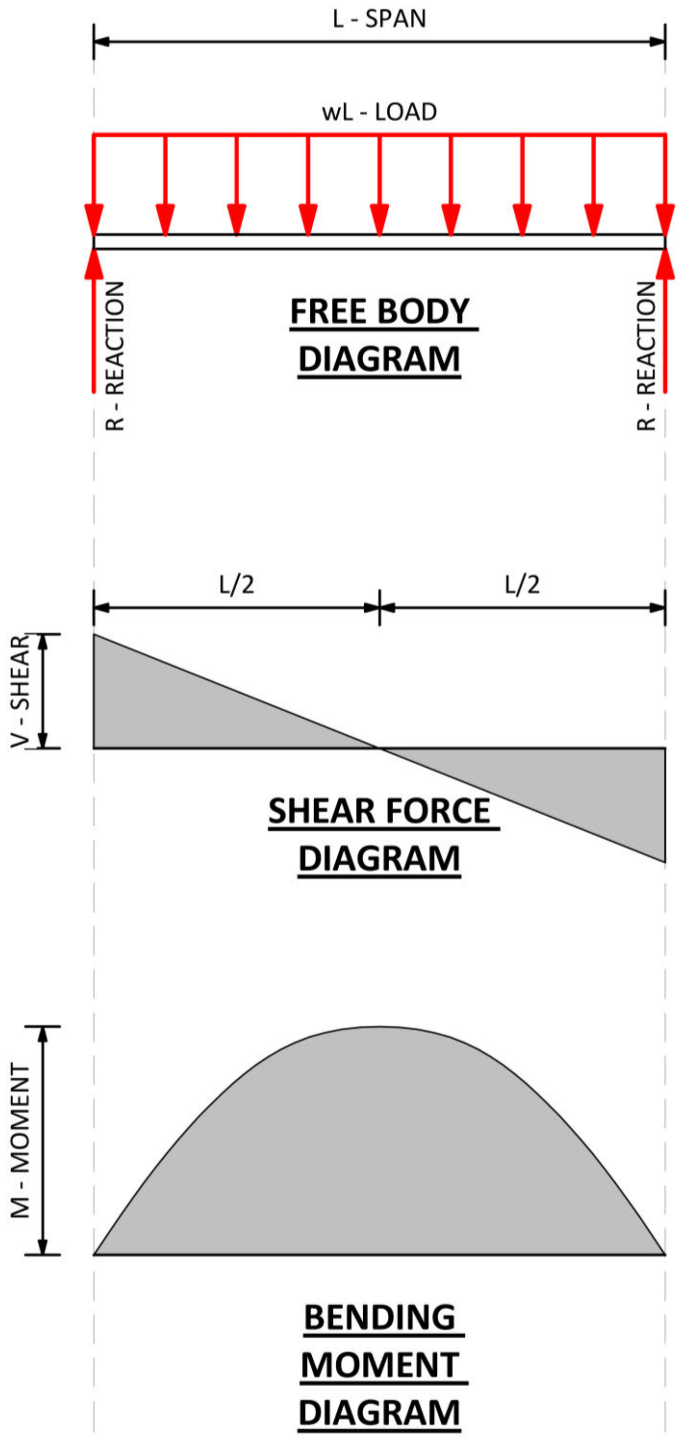

Figure 7.2 - Uniformly Distributed Load Force Diagrams

The force diagrams, along with the established equations reflect the expected conditions for a uniformly distributed loaded sandwich panel beam in a similar manner to section 5.2.2 of this report. This approach will allow further understanding of the stresses that the test samples were subject to during the test and at the eventual failure point.

$$\mathrm{R} = \mathrm{V}$$

$$\mathrm{V} = \frac{\mathrm{wL}}{2}$$

$$\mathrm{M} = \frac{\mathrm{wL}^2}{8}$$

Where:

$\mathrm{R} = \text{Reaction at support (kN)}$

$\mathrm{V} = \text{Shear Force (kN)}$

$\mathrm{M} = \text{Moment Force (kN.m)}$

$\mathrm{w} = \text{Applied Load (kN/m)}$

$\mathrm{L} = \text{Span (m)}$

The failure modes that will be considered for the predictive analysis of the ultimate strength conditions will be shear failure of the core and bending failure of the skin material, as presented in section 5.2.2 of this report. As discussed in section 5.2.1.4 of this report, bending of the core material does not need to be considered as the skin material is providing the required bending resistance.

The summary of bending moment capacities that correspond with each core thickness, core density and skin thickness that will be investigated and utilised for the span table calculation can be seen in Table 7.3 (no topping slab present hence neutral axis is centrally located for wall, roof and floor applications) and Table 7.4 (where topping slab is present hence neutral axis is not centrally located and used for the formwork applications after stage 3 of the construction cycle) below.

This data will be used when determining the ultimate strength requirements for the bending capacity of each application. The reduction factor (Ø) outlined in section 6.3 of this report for the steel skin material in bending is 0.9 as determined from the Australian Standards and will be used in the analysis and subsequent capacity values giving in the span tables.

Table 7.3 - Bending Moment Capacity Values for Combinations (No Topping Slab)

Bending Moment Capacities – No Topping Slab (kN.m)					
Core Thickness	Core Density (kg/m^3)	Skin Thickness (mm)			
		0.40	0.55	0.95	1.95
75mm	400	5.8	7.7	12.6	25.3
	510	6.1	7.9	12.9	25.5
	550	6.2	8.1	13.0	25.6
	580	6.3	8.1	13.1	25.7
100mm	400	8.2	10.7	17.2	33.9
	510	8.7	11.1	17.7	34.3
	550	8.9	11.3	17.9	34.5
	580	9.0	11.5	18.0	34.7
125mm	400	10.8	13.9	22.1	42.8
	510	11.5	14.5	22.7	43.4
	550	11.8	14.9	23.1	43.8
	580	12.1	15.1	23.3	44.0
150mm	400	16.9	20.6	30.4	55.1
	510	17.9	21.5	31.3	56.0
	550	18.4	22.0	31.8	56.5
	580	18.7	22.4	32.2	56.9
175mm	400	21.2	25.5	36.9	65.7
	510	22.6	26.8	38.2	67.0
	550	23.2	27.5	38.9	67.6
	580	23.7	28.0	39.4	68.1
200mm	400	25.8	30.7	43.7	76.5
	510	27.6	32.4	45.5	78.3
	550	28.4	33.3	46.3	79.1
	580	29.1	33.9	47.0	79.8

Table 7.4 - Bending Moment Capacity Values for Combinations (With Topping Slab)

Bending Moment Capacities – With Topping Slab (kN.m)					
Core Thickness	Core Density (kg/m^3)	Skin Thickness (mm)			
		0.40	0.55	0.95	1.95
75mm	400	19.9	22.9	31.0	51.0
	510	20.9	24.0	32.0	52.0
	550	21.4	24.5	32.5	52.4
	580	21.8	24.9	32.9	52.8
100mm	400	23.1	26.6	36.0	59.4
	510	24.6	28.1	37.5	60.8
	550	25.4	28.9	38.3	61.4
	580	25.9	29.5	38.8	62.0
125mm	400	28.2	32.2	43.0	69.7
	510	30.3	34.3	45.1	71.7
	550	31.3	35.4	46.1	72.6
	580	32.1	36.1	46.8	73.3
150mm	400	37.3	41.9	54.0	84.2
	510	40.1	44.6	56.7	86.7
	550	41.4	45.9	58.0	88.0
	580	42.4	46.9	59.0	88.9
175mm	400	42.8	47.9	61.5	95.4
	510	46.3	51.4	64.9	98.6
	550	48.0	53.1	66.6	100.2
	580	49.3	54.4	67.8	101.4
200mm	400	48.8	54.5	69.6	107.2
	510	53.1	58.8	73.8	111.2
	550	55.3	60.9	75.9	113.2
	580	56.9	62.5	77.4	114.7

The summary of core shear capacities that correspond with each core thickness and core density that will be investigated and utilised for the span table calculation can be seen in Table 7.5 below. This data will be used when determining the ultimate strength requirements for the shear capacity of each application. The reduction factor (Ø) outlined in section 6.3 of this report for the AAC core material in shear is 0.7 as determined from the Australian Standards and will be used in the analysis and subsequent capacity values giving in the span tables.

Table 7.5 - Shear Capacity Values for Combinations

Shear Capacities (kN)		
Core Thickness	**Core Density (kg/m^3)**	**Shear Capacity (kN)**
75mm	**400**	3.5
	510	4.1
	550	5.9
	580	5.9
100mm	**400**	4.7
	510	5.5
	550	7.8
	580	7.8
125mm	**400**	5.9
	510	6.9
	550	9.8
	580	9.8
150mm	**400**	7.1
	510	8.2
	550	11.8
	580	11.8
175mm	**400**	8.2
	510	9.6
	550	13.7
	580	13.7
200mm	**400**	9.4
	510	11.0
	550	15.7
	580	15.7

7.4 Span Tables

In order to show the load capacity data in a format which can best display the capabilities of each explored arrangement and application, span tables and corresponding graphing will be generated. An example of the span table output can be seen in Table 7.6 below, with the key factors being an increase of the core thickness (in mm) going down the rows and increasing span length (in m) across the columns. The table also displays other constant information for each table such as the intended application, the sheet thickness, the panel width and core density.

For the wall and roof applications only, the yellow (N1), blue (N3) and green (N5) shading indicates the maximum wind classification region that each particular arrangement can be used in whilst still adhering to the outlined criteria. For all applications the red shading indicates that the specific arrangement does not meet the criteria and therefore is not a useable arrangement and the lettering indicates which factor/s is limiting the use. If the serviceability deflection criteria are not met, the letter D is substituted in lieu of a kPa capacity and when it is a strength failure of core shear of bending of the skin material, a C and S will be shown respectively. An F is shown in the formwork span tables if the deflection from the formwork loading exceeds the allowable amount. Multiple lettering indicates that the specimen fails more than one of the criteria.

Lastly, for each of the three failures (deflection, shear and bending) the percentage of how often each one provides the lowest capacity and is therefore limiting the capacity of the system first. This will provide a guide as to which component is causing the majority of the failures, and will lead to recommendations on the skin thickness and core density.

Table 7.6 - Example Span Table

Example Maximum Load Capacity (kPa) Table **Single Span – Limited by Serviceability & Ultimate Limit State**										
Sheet Thickness	**mm**	**N1 Region Max**	**Limiting**	**Deflection (D)**	**%**					
Panel Width	**mm**	**N3 Region Max**		**Shear (C)**	**%**					
Core Density	**kg/m^3**	**N5 Region Max**		**Bending (S)**	**%**					
Core Thickness	**Span Length (m)**									
	1.5	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**	**6**
75mm										
100mm										
125mm										
150mm										
175mm										
200mm										

7.4.1 Decision-Making Process

For each potential application, the serviceability and strength criteria that is associated with them differs, as discussed throughout this chapter. For each application, a decision-making flow chart has been generated to graphically show the process that each cell in the table was based upon. The differences in criteria change the outcomes and results, which will be discussed in section 8 of this report.

The justification for providing the capacity in maximum kPa uniformly distributed pressure load as opposed to a more simple approach which purely states whether the arrangement works or not, is so that potential designers can still apply the span tables for their own specific loading conditions, as the highly variable nature of design cannot be covered in a single way. This is particularly evident for the wall and roof applications, where the span tables intend to provide a generic guide on which wind region each arrangement can be appropriately used in. It would be the responsibility of each design engineer to identify if their particular circumstances would be within or outside of the loading capacity given in the span tables, though the data is presented in such a way that a guide on its capabilities are given.

7.4.1.1 Wall

Figure 7.3 shows the decision-making process for strength and serviceability for the walling application.

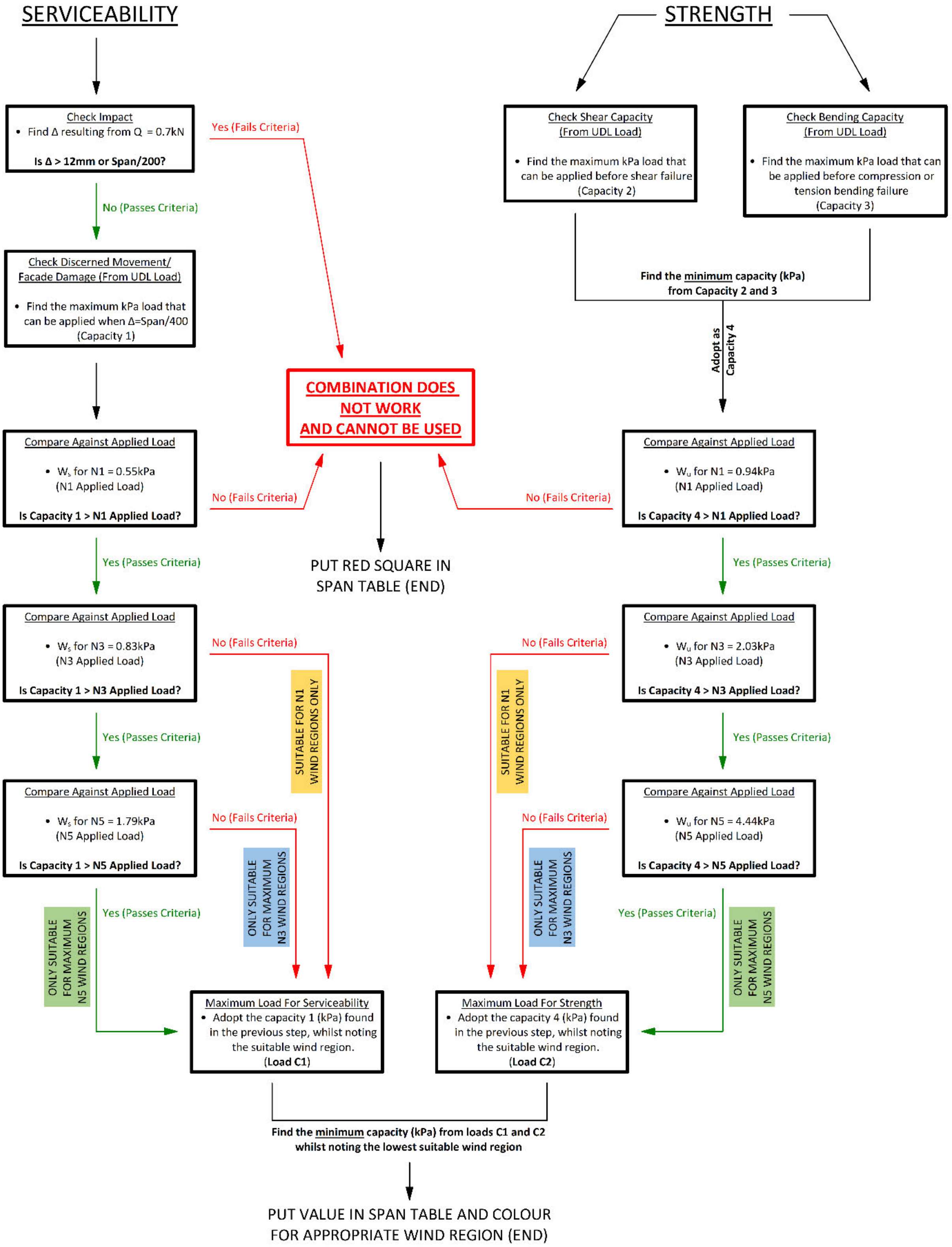

Figure 7.3 - Decision Making Flowchart - Wall

7.4.1.2 Roof

Due to the complexity of the factors involved in the roofing applications, the serviceability (Figure 7.4) and strength (Figure 7.5) decision making flow charts have split into two different diagrams, and later combine to consider both states (Figure 7.6).

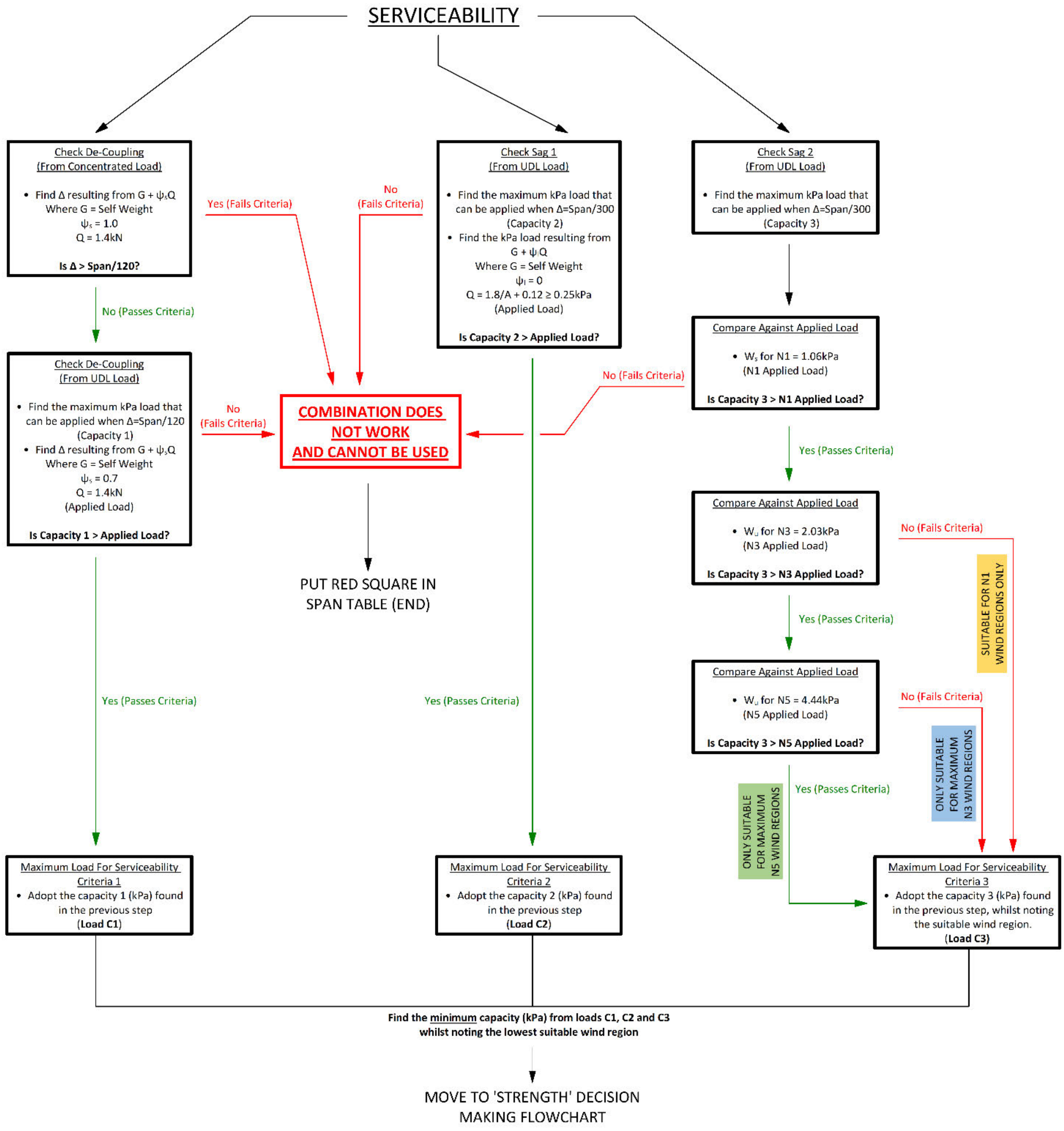

Figure 7.4 - Serviceability Decision Making Flowchart - Roof

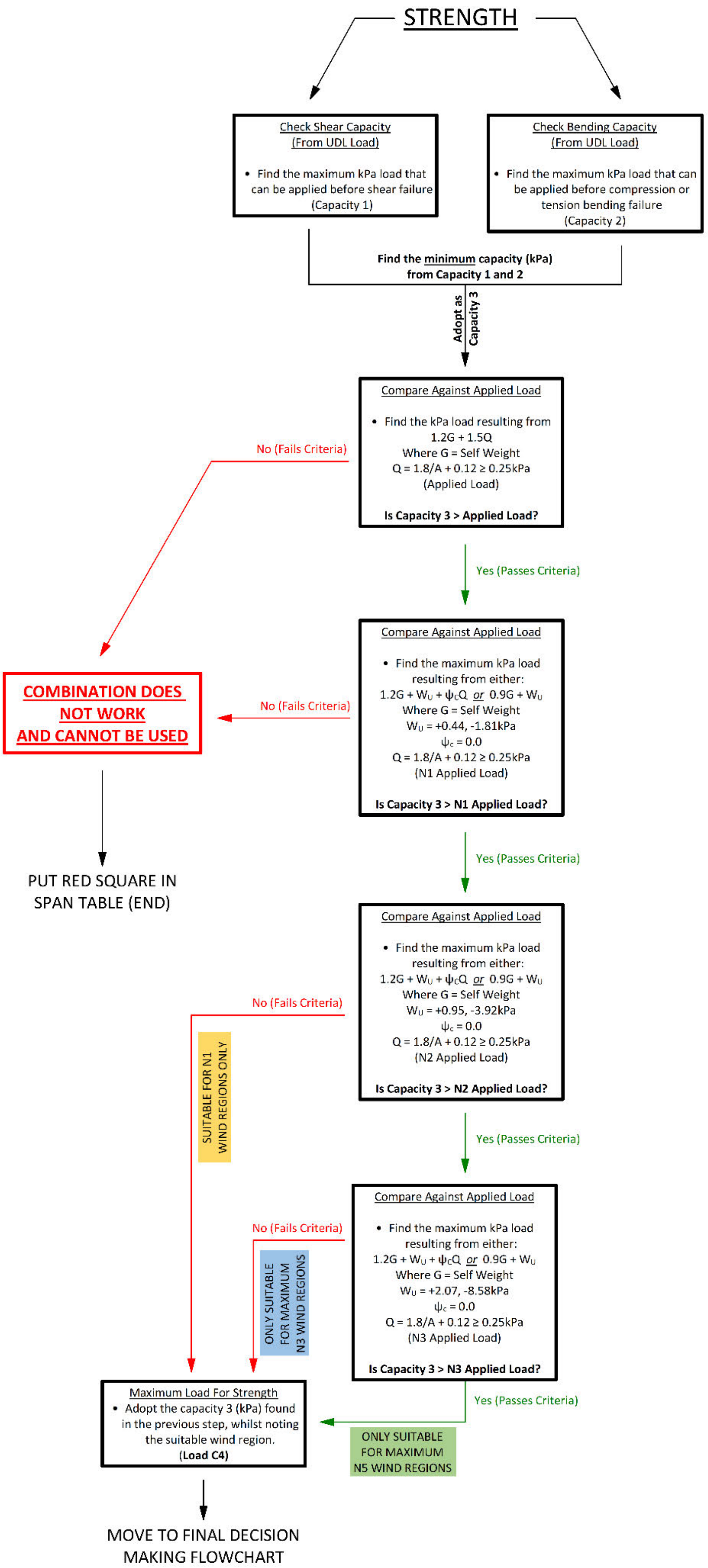

Figure 7.5 - Strength Decision Making Flowchart - Roof

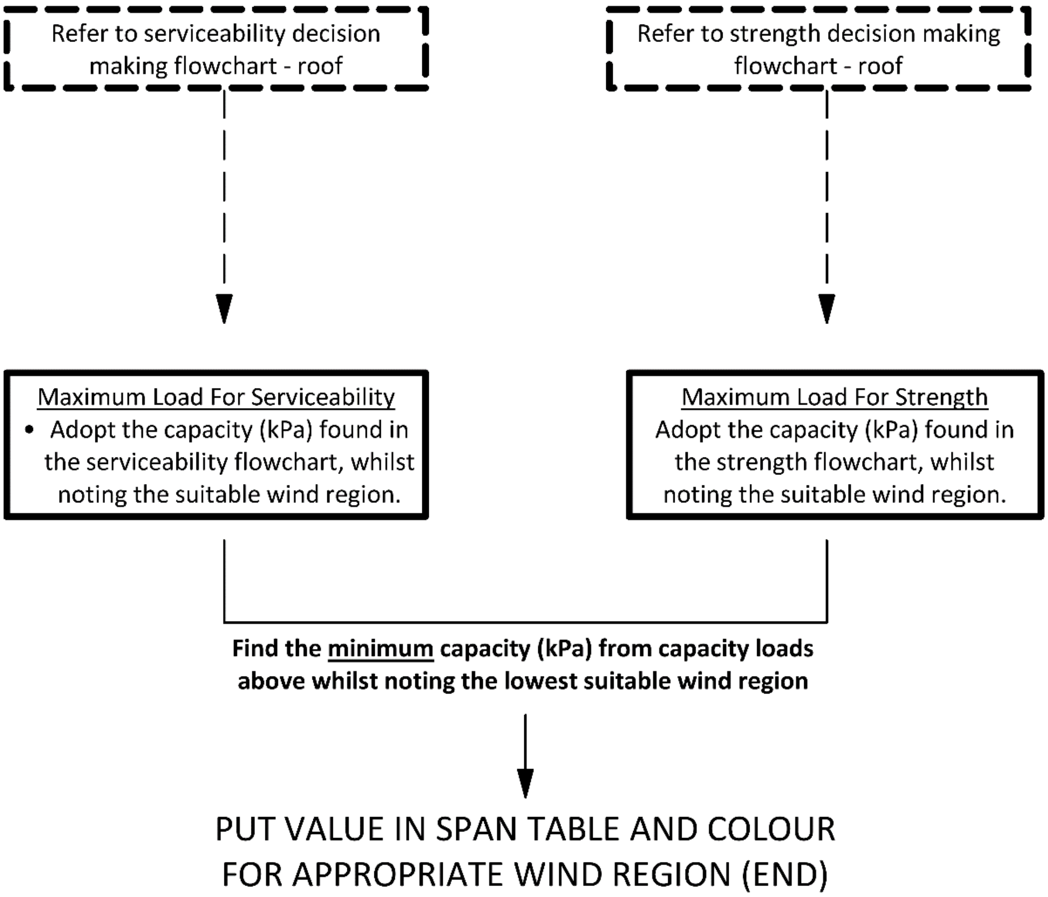

Figure 7.6 - Final Decision-Making Flowchart - Roof

7.4.1.3 Floor

Figure 7.7 shows the decision-making process for strength and serviceability for the floor application, which when compared to that used for a wall or roof application is simper due to wind regions not being a factor therefore more consistent loading cases are presented.

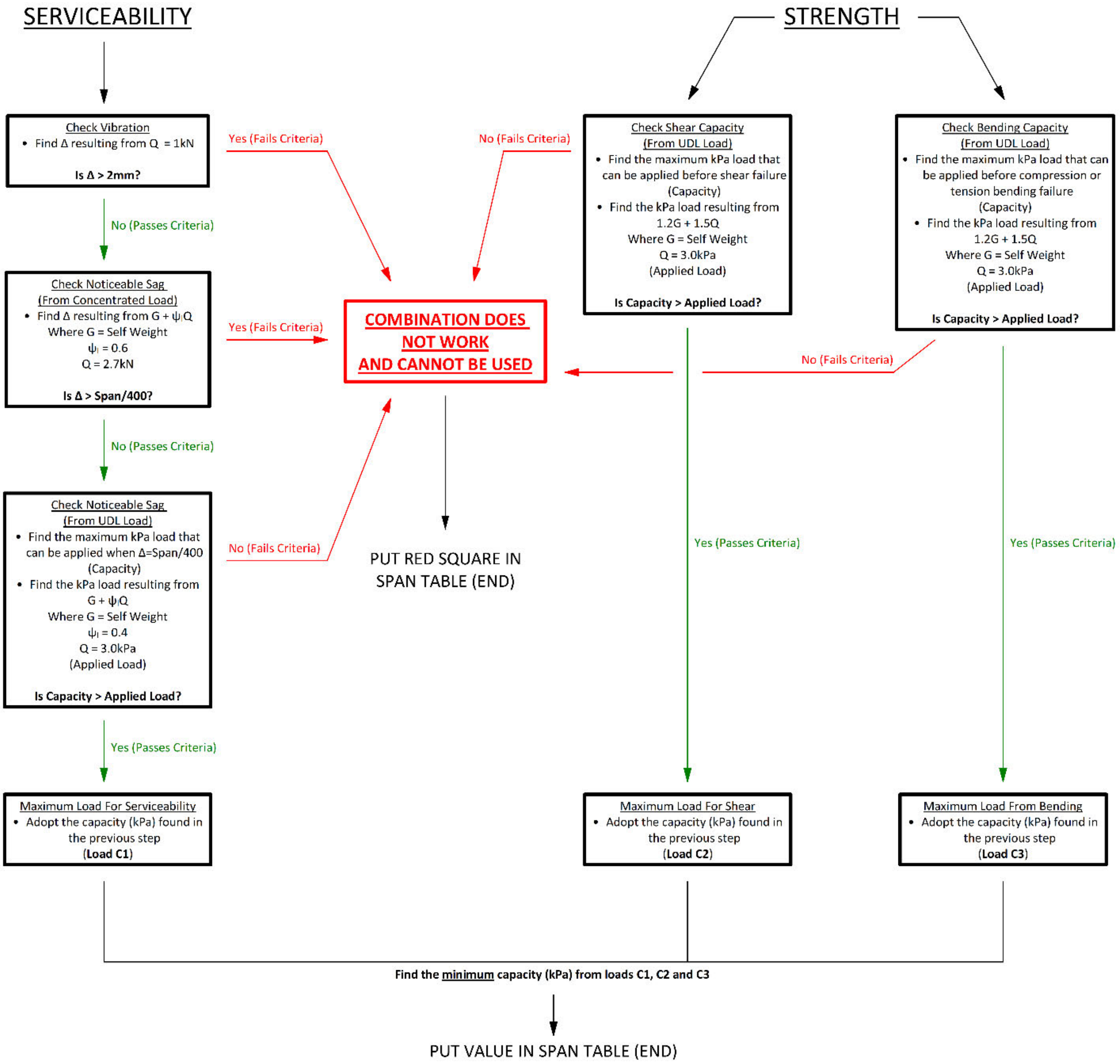

Figure 7.7 - Decision Making Flowchart - Floor

7.4.1.1 Formwork

Figure 7.8 shows the decision-making process for strength and serviceability for the floor application, which considers the loading required from a formwork system. Some elements of the chart relate to the floor application chart as a result of the permanent nature of the system.

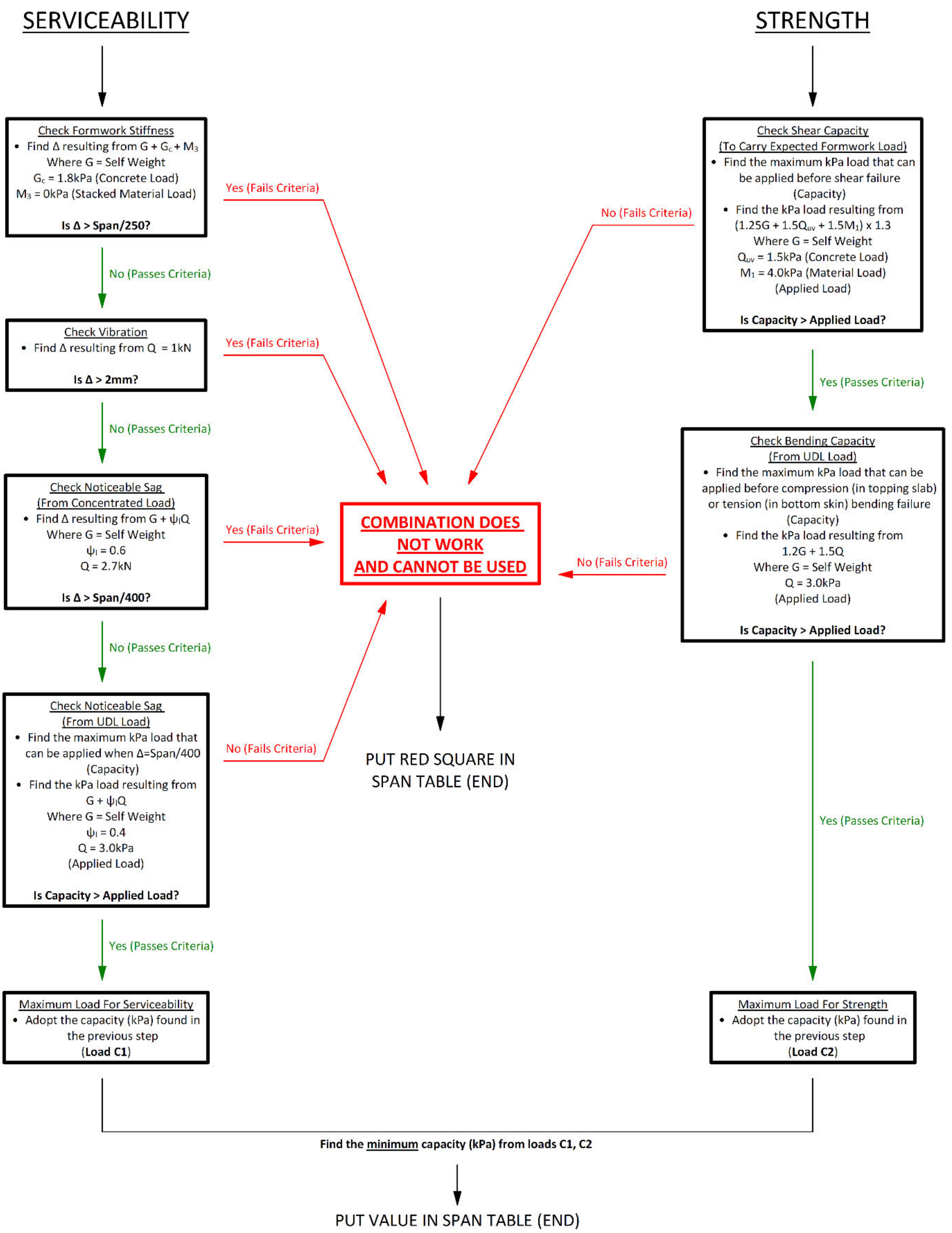

Figure 7.8 - Decision Making Flowchart - Formwork

8 Results & Discussion

Using the criteria outlined in section 6 of this report, along with the analytical methods first established in section 5 and further adapted in section 7, the decision-making processing charts were successfully applied to produce span tables for every configuration of building application and variation of core density and steel skin thickness.

Based on the testing and subsequent theory analysis, it has been established that the AAC core is carrying the shear forces and the sandwich configuration of the steel skins are contributing to the bending resistance. Further discussion on how the core and skin materials impact the system are presented in the following sections.

The results are broken down into each building application with a subsequent discussion explaining the findings and how all of the factors involved effected the results found.

8.1 Core Material

With the core material carrying the shear forces, it was seen (as expected) that increasing the density of the core provides an increase in the shear carrying capacity of the system, whilst increasing the thickness of the core also has a beneficial effect. It was found that another advantage of having a thicker core segment is that it increases the distance between the steel skin materials and therefore helps to increase the overall flexural rigidity and hence increase the bending resistance provided and helps the system meet the serviceability criteria.

8.2 Skin Material

For the reasons discussed in section 4.4.3, steel was chosen as the skin material and during the analysis it can be seen that increasing the thickness has a significant effect on the resistance of deflection and is the largest contributing factor for satisfying the serviceability criteria. It was also found that the self-weight of the system is affected in such a way that in some cases (particularly the horizontally orientated applications), the additional weight resulting from the thicker sheets limits some of the spans when compared to the thinner options.

8.3 Building Application

It was seen that each building application has a large variance on the serviceability criteria and also the expected loading conditions, which had a large bearing on the results and capabilities of each system.

In terms of serviceability, it was seen that the wall application was the most capable due to its less stringent deflection limits, combined with lower expected loading conditions when compared to the other building applications. The serviceability criteria for the roofing application was not met as frequently as the wall application as a result and the flooring was seen to be even more restrictive again as the vertical loading is more prevalent and the allowable deflection is also small. It was seen that many configurations that combined a thin core with long span distance being found to be not a valid or useable solution, often in the top right-hand corner of the tables. The failures were indicated by the red shading and lettering in the span tables as discussed in section 7.4 of this report.

The ultimate strength analysis presented findings similar to what was seen from the serviceability calculations, in terms of an increase in expected loading in the order of wall, roof and the highest being the floor application. This was again reflected in the span tables, with most wall configurations usable in higher wind regions when comparing the same specification to that of a roof application. The floor application was seen be even more limited in its use when compared to the wall and roof applications and this was due to the larger vertical loads it is subject to. The formwork presented much different outcomes to the other three applications, as the construction loads were significant enough to predominantly be the loading application that produced the most adverse conditions, particularly when considering the shear strength of the core during the formwork construction phase.

8.4 Failures

Lastly, for each of the three failures (deflection, shear and bending) the percentage of how often each one provides the lowest capacity and is therefore limiting the capacity of the system first. The shear failures are put down to the core material as a percentage and as the skin material has the largest bearing on the deflection and bending failures and they are weighted for the skin material causing the failure.

The breakdown of all the combinations are given in tabulated format at the beginning of each section. This provides a guide for which component is causing the majority of the failures and helps to present which combinations of core density and skin thickness is efficient and which combinations are not. For example, if the skin material is the limiting (first cause of failure) factor 80% of the time, then the data suggests that the skin is not thick enough and also that the core material is essentially being underutilised and therefore stronger than it ideally would be to create an efficient design. The combinations that provide limiting percentages better than a 40% to 60% split will be considered to be efficient and therefore highlighted as green. For the available efficient designs, the best outcomes will be present as solutions in this results chapter, with the remaining span tables to be provided in full in the relevant appendices (Appendix K, Appendix L, Appendix M and Appendix N for wall, roof, floor and formwork respectively). This is not indicating that the remaining combinations are not usable, it's just simply not as efficient as the solutions presented and discussed in this chapter.

8.5 Wall

The analysis of the walling application was undertaken using the methods outlined in the previous chapters, with the failure of each material being used to quantify the efficiency and therefore optimised combinations. Every span table and graph for the wall application is given in Appendix K and a summary given in Table 8.1. This section shows the breakdown of all wall combinations in terms of core densities and skin thickness, showing how often each respective material is limiting the system as a percentage. A breakdown (as a percentage) is also given for how much of the span table cannot be used at all, and how many of the available combinations can be used in N1, N3 and N5 are also shown.

The following makeup of AAC core density and steel skin thicknesses were found to be the recommended arrangements for a wall application:

- **510kg/m^3 AAC core with 0.40mm steel skin** (Refer to Table 8.2 and Figure 8.1)
- **550kg/m^3 AAC core with 0.55mm steel skin** (Refer to Table 8.3 and Figure 8.2)

Other findings from the wall analysis include:

- Core density of 400kg/m^3 is not recommended to be used, as even the thinnest considered skin option is being underutilised with a maximum limiting failure percentage of 40%.
- Core density of 580kg/m^3 is not recommended to be used as despite having a better ratio for the material failures when compared to the 550kg/m^3 option (53.3%-46.7% against 51.7%-48.3%), it does not provide an significant increase in performance and therefore adds unnecessary additional weight to the system.
- Skin thicknesses of 0.95mm and 1.95mm are excessively thick for the expected loading conditions for a wall.
- The thicker cores of 175mm and 200mm provide capacity that is well above the expected loading conditions for a wall, even when considering N5 wind region. Therefore, the 150mm core is the largest recommended core thickness for the wall applications being assessed in this dissertation.

Table 8.1 - Wall Limiting Results Summary

Limiting Component Summary							
Application	Core Density (kg/m^3)	Failure		Skin Thickness (mm)			
				0.40	0.55	0.95	1.95
Wall	400	Skin Limiting Core Limiting		40% 60%	31.7% 68.3%	16.7% 83.3%	1.7% 98.3%
		Usable Wind Region	Not Usable	3.3%	1.7%	0.0%	0.0%
			N1 Max	3.3%	1.7%	1.7%	1.7%
			N3 Max	23.3%	26.7%	28.3%	28.3%
			N5 Max	70.0%	70.0%	70.0%	70.0%
	510	Skin Limiting Core Limiting		46.7% 53.3%	40% 60%	21.7% 78.3%	3.3% 96.7
		Usable Wind Region	Not Usable	3.3%	1.7%	0.0%	0.0%
			N1 Max	3.3%	1.7%	0.0%	0.0%
			N3 Max	13.3%	16.7%	20.0%	20.0%
			N5 Max	80.0%	80.0%	80.0%	80.0%
	550	Skin Limiting Core Limiting		56.7% 46.3%	53.3% 46.7%	35% 65%	11.7% 88.3%
		Usable Wind Region	Not Usable	3.3%	1.7%	0.0%	0.0%
			N1 Max	3.3%	1.7%	0.0%	0.0%
			N3 Max	10.0%	10.0%	8.3%	8.3%
			N5 Max	83.3%	86.7%	91.7%	91.7%
	580	Skin Limiting Core Limiting		56.7% 46.3%	51.7% 48.3%	35% 65%	11.7% 88.3%
		Usable Wind Region	Not Usable	3.3%	1.7%	0.0%	0.0%
			N1 Max	3.3%	1.7%	0.0%	0.0%
			N3 Max	10.0%	10.0%	8.3%	8.3%
			N5 Max	83.3%	86.7%	91.7%	91.7%

Table 8.2 - Span Table - Wall - 510kg/m3 - 0.40mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)	46.7	%			
Panel Width	600	mm	N3 Region Max		Shear (C)	53.3	%			
Core Density	510	kg/m³	N5 Region Max		Bending (S)	0.0	%			
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	3.2	2.0	1.4	1.0	0.7	D	D
100mm	12.2	9.1	7.3	6.1	3.8	2.6	1.8	1.3	1.0	0.8
125mm	15.2	11.4	9.1	7.6	6.4	4.3	3.0	2.2	1.6	1.3
150mm	18.3	13.7	11.0	9.1	7.8	6.9	5.6	4.1	3.1	2.4
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.0	4.5	3.5
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.3	4.8

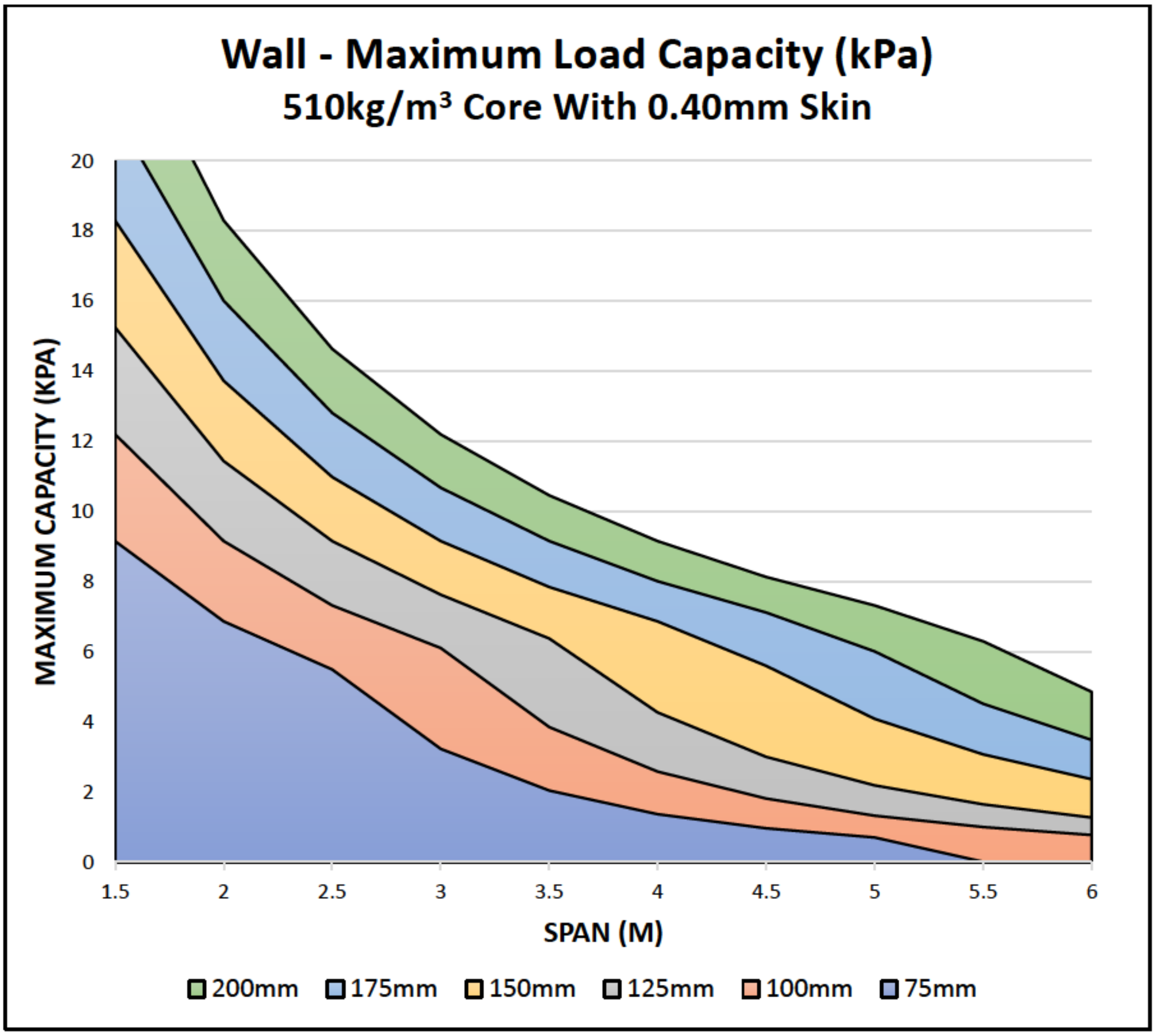

Figure 8.1 - Load Capacity Graph - Wall - 510kg/m³ - 0.40mm

Table 8.3 - Span Table - Wall - 550kg/m3 - 0.55mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)		53.3	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		46.7	%		
Core Density	550	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.4	4.3	2.7	1.8	1.3	0.9	0.7	D
100mm	17.4	13.1	10.5	8.0	5.0	3.4	2.4	1.7	1.3	1.0
125mm	21.8	16.3	13.1	10.9	8.3	5.5	3.9	2.8	2.1	1.6
150mm	26.1	19.6	15.7	13.1	11.2	9.8	6.9	5.0	3.8	2.9
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.0	7.3	5.5	4.2
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.1	7.6	5.9

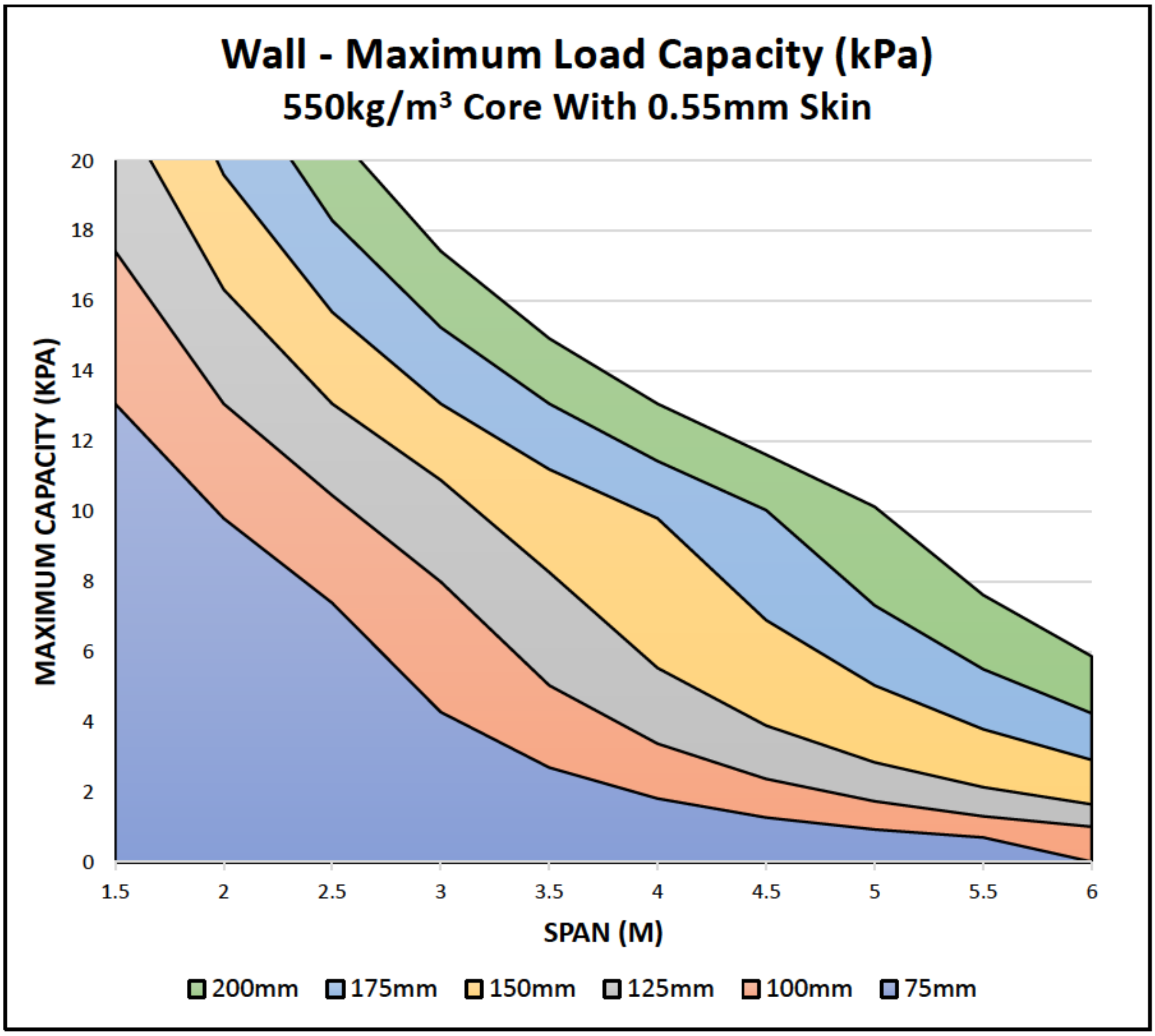

Figure 8.2 - Load Capacity Graph - Wall - 550kg/m3 - 0.55mm

8.6 Roof

Applying the methodology developed through this dissertation, the analysis of the roof application was completed to develop all of the span tables and graphs for each configuration. All of which are presented in Appendix L of this report and summarised in Table 8.4 in terms of failures and capabilities. The relevant shading is also used to represent the applicable wind regions in a similar fashion to the previous wall section.

The following makeup of AAC core density and steel skin thicknesses were found to be the recommended arrangements for a wall application:

- **400kg/m^3 AAC core with 0.40mm steel skin** (Refer to Table 8.5 and Figure 8.3)
- **510kg/m^3 AAC core with 0.55mm steel skin** (Refer to Table 8.6 and Figure 8.4)
- **550kg/m^3 AAC core with 0.95mm steel skin** (Refer to Table 8.7 and Figure 8.5)

Other findings from the roof analysis include:

- Using an AAC core with a density of 580kg/m^3 is not recommended as it does not provide a significant increase in performance and therefore adds unnecessary additional weight to the system.
- Having a skin thickness of 1.95mm is excessively thick for the expected loading conditions for a roof.

Table 8.4 - Roof Limiting Results Summary

Limiting Component Summary							
Application	**Core Density (kg/m³)**	**Failure**		**Skin Thickness (mm)**			
				0.40	**0.55**	**0.95**	**1.95**
Roof	**400**	Skin Limiting Core Limiting		50% 50%	40% 60%	21.7% 78.3%	5% 95%
		Usable Wind Region	Not Usable	13.3%	10%	3.3%	0.0%
			N1 Max	6.7%	6.7%	13.3%	15.0%
			N3 Max	53.3%	55.0%	55.0%	56.7%
			N5 Max	26.7%	28.3%	28.3%	28.3%
	510	Skin Limiting Core Limiting		55% 45%	45% 55%	30% 70%	8.3% 91.7%
		Usable Wind Region	Not Usable	11.7%	8.3%	3.3%	0.0%
			N1 Max	6.7%	6.7%	6.7%	8.3%
			N3 Max	41.7%	45.0%	50.0%	51.7%
			N5 Max	40.0%	40.0%	40.0%	40.0%
	550	Skin Limiting Core Limiting		65% 35%	56.7% 43.3%	45% 55%	18.3% 81.7%
		Usable Wind Region	Not Usable	10.0%	8.3%	3.3%	0.0%
			N1 Max	8.3%	6.7%	3.3%	1.7%
			N3 Max	15.0%	18.3%	26.7%	31.7%
			N5 Max	66.7%	66.7%	66.7%	66.7%
	580	Skin Limiting Core Limiting		65% 35%	55% 45%	45% 55%	18.3% 81.7%
		Usable Wind Region	Not Usable	10.0%	6.7%	3.3%	0.0%
			N1 Max	8.3%	8.3%	3.3%	1.7%
			N3 Max	15.0%	18.3%	26.7%	31.7%
			N5 Max	66.7%	66.7%	66.7%	66.7%

Table 8.5 - Span Table - Roof - 400kg/m³ - 0.40mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.40 mm	N1 Region Max	Limiting	Deflection (D) 50.0 %							
Panel Width 600 mm	N3 Region Max		Shear (C) 50.0 %							
Core Density 400 kg/m³	N5 Region Max		Bending (S) 0.0 %							
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.5	2.6	1.6	1.1	D	D	D	D
100mm	10.5	7.8	6.3	4.8	3.0	2.0	1.4	D	D	D
125mm	13.1	9.8	7.8	6.5	5.0	3.3	2.4	1.7	1.3	D
150mm	15.7	11.8	9.4	7.8	6.7	5.9	4.4	3.2	2.4	1.9
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	4.7	3.5	2.7
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	4.9	3.8

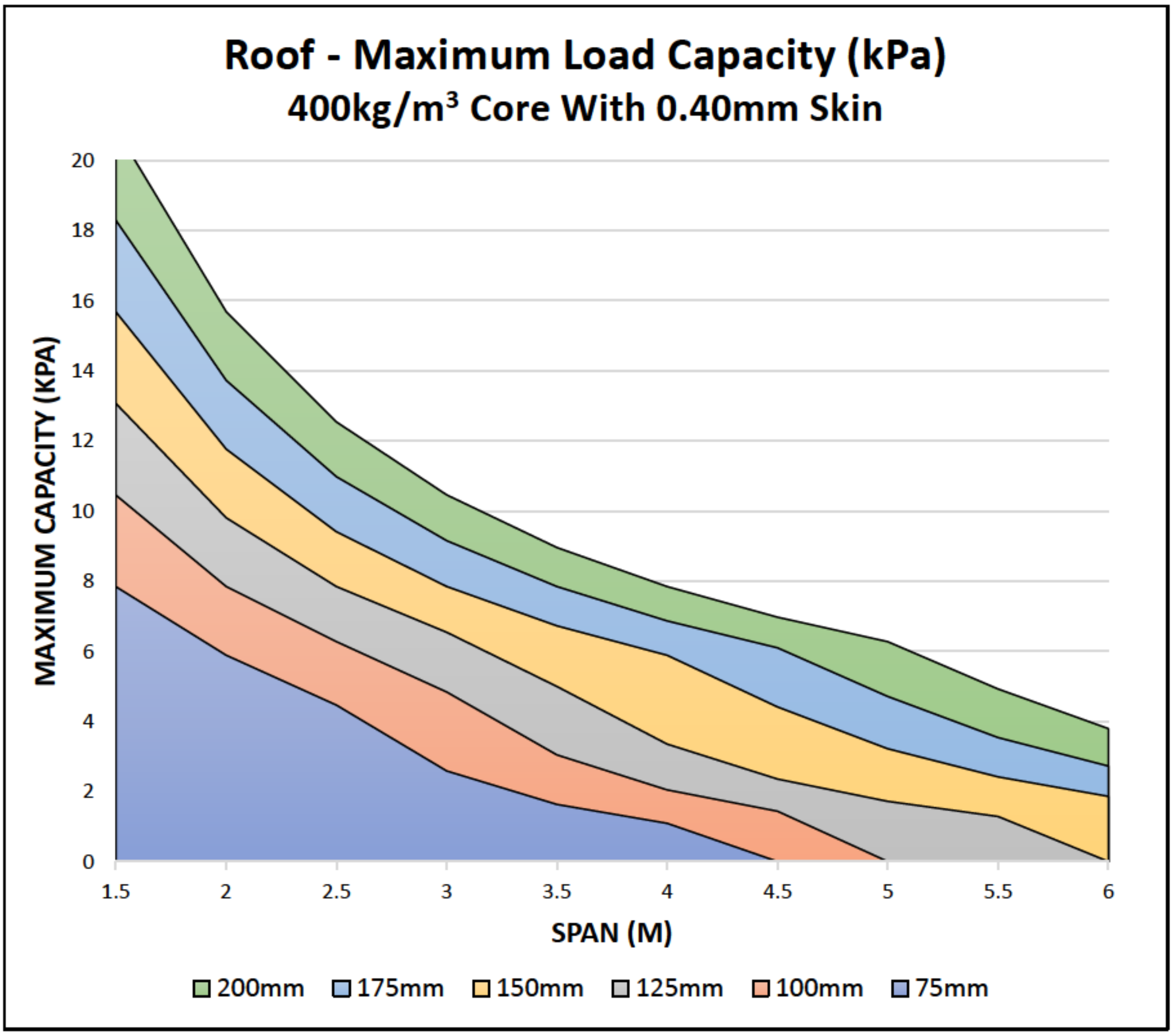

Figure 8.3 - Load Capacity Graph - Roof - 400kg/m3 - 0.40mm

Table 8.6 - Span Table - Roof - 510kg/m³ - 0.55mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)	45.0	%			
Panel Width	600	mm	N3 Region Max		Shear (C)	55.0	%			
Core Density	510	kg/m³	N5 Region Max		Bending (S)	0.0	%			
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	3.5	2.2	1.5	D	D	D	D
100mm	12.2	9.1	7.3	6.1	4.1	2.8	1.9	1.4	1.1	D
125mm	15.2	11.4	9.1	7.6	6.5	4.5	3.2	2.3	1.7	1.3
150mm	18.3	13.7	11.0	9.1	7.8	6.9	5.6	4.1	3.1	2.4
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.0	4.5	3.4
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.2	4.8

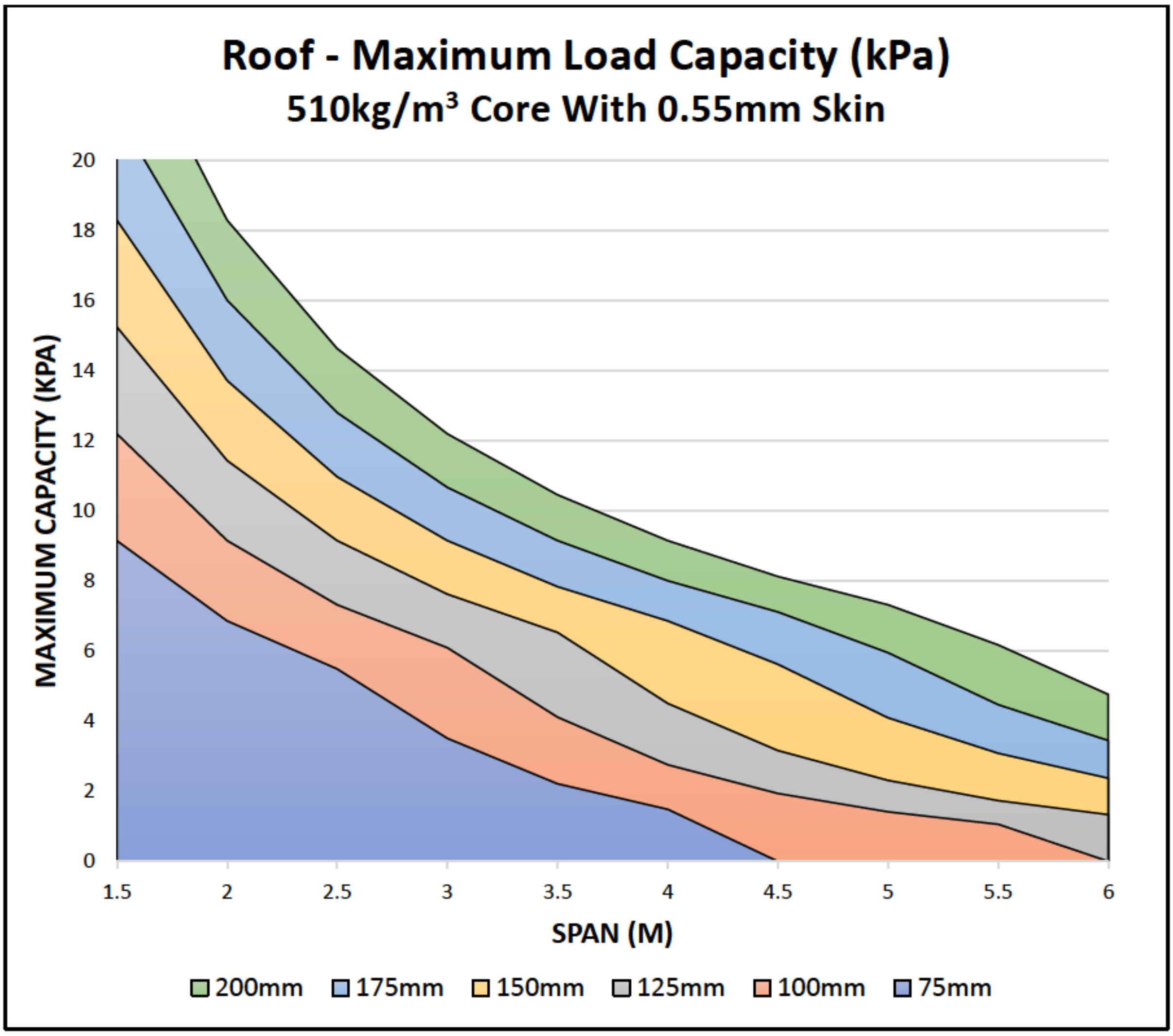

Figure 8.4 - Load Capacity Graph - Roof - 510kg/m3 - 0.55mm

Table 8.7 - Span Table - Roof - 550kg/m³ - 0.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max		Limiting	Deflection (D)		45.0	%	
Panel Width	600	mm	N3 Region Max			Shear (C)		55.0	%	
Core Density	550	kg/m³	N5 Region Max			Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	5.8	3.6	2.4	1.7	1.2	D	D
100mm	17.4	13.1	10.5	8.7	6.7	4.5	3.1	2.3	1.7	1.3
125mm	21.8	16.3	13.1	10.9	9.3	7.2	5.0	3.7	2.8	2.1
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.3	6.1	4.6	3.5
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	8.6	6.5	5.0
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	8.8	6.8

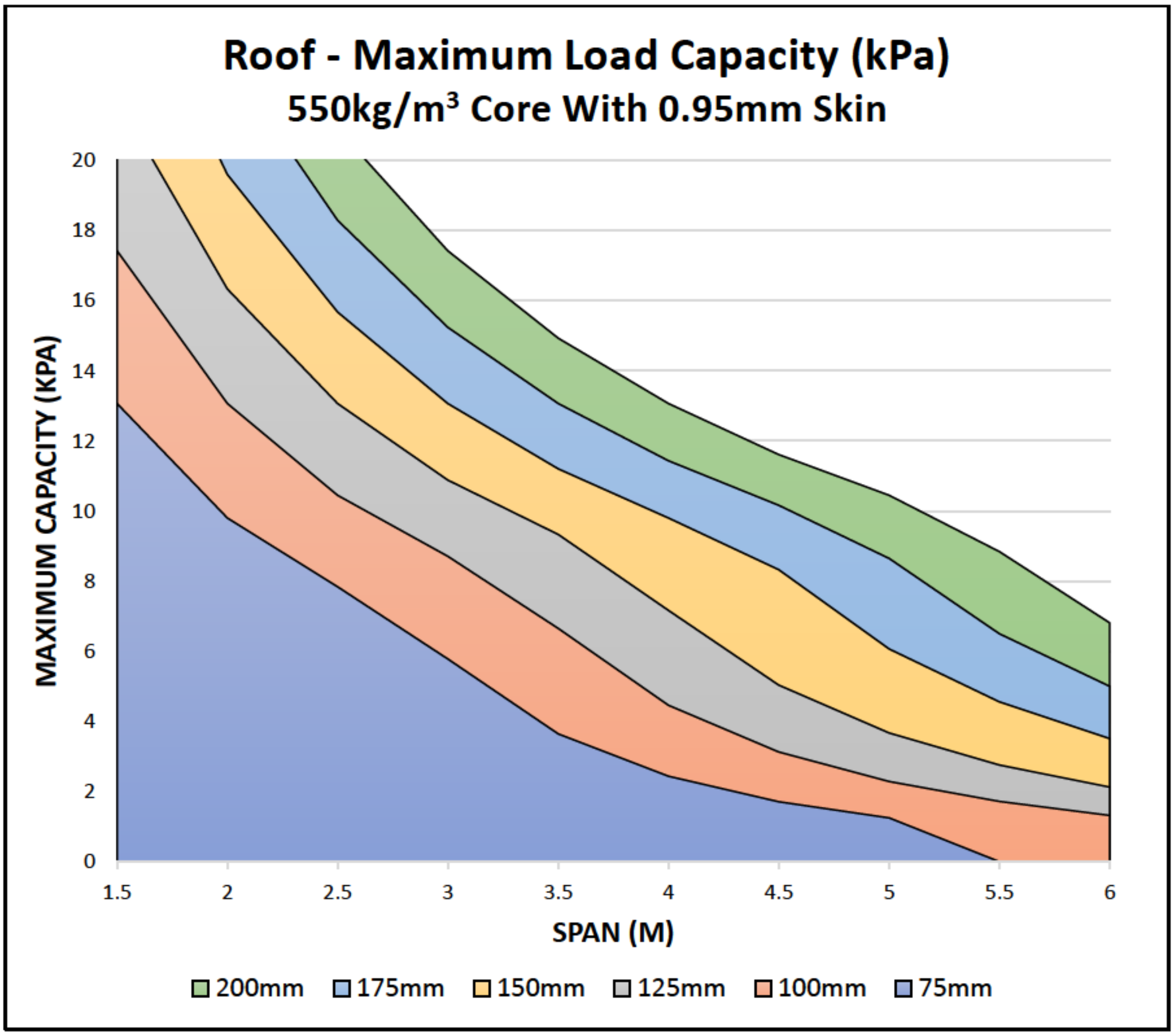

Figure 8.5 - Load Capacity Graph - Roof - 550kg/m3 - 0.95mm

8.7 Floor

The development of the analysis through this dissertation allows span tables and graphs to be developed for flooring applications which can be seen for all variables covered in the Appendix M. As discussed in section 8.3, the flooring application was seen to be the building application with the most stringent deflection limits and generally high expected loading conditions in comparison to the wall and roofing scenarios. As a result, it is a general finding that thicker skins, thicker cores, denser cores or a combination of all three are required to achieve sufficient structural performance as a floor when compared to the aforementioned building applications. A full summary of the material influence and the pass/fail percentages are provided in Table 8.8, from which the best solutions are chosen from.

The following makeup of AAC core density and steel skin thicknesses were found to be the recommended arrangements for a floor application:

- **510kg/m^3 AAC core with 0.55mm steel skin** (Refer to Table 8.9 and Figure 8.6)
- **550kg/m^3 AAC core with 0.95mm steel skin** (Refer to Table 8.10 and Figure 8.7)

Other findings from the wall analysis include:

- A core density of 400kg/m^3 can be used in terms of an efficient combination with a sheet thickness of 0.55mm, though it is recommended (as above) that an increase in the core density to 510kg/m^3 is advised as this provides an increase of passing combinations of 10%
- As per the wall and roof applications, having a core density of 580kg/m^3 is not recommended to be used as there is an insignificant increase in performance, and also adds unnecessary additional weight to the system.
- Using a skin thickness of 0.40mm is too thin for a floor application, as causes the first failure at least 60% of time.
- 1.95mm is too thick to be used for the floor application, as the ratio of failures between the skin and core become inefficient as the shear failures seen in the core increase.

Table 8.8 - Floor Limiting Results Summary

Limiting Component Summary						
Application	**Core Density (kg/m^3)**	**Failure**	**Skin Thickness (mm)**			
			0.40	**0.55**	**0.95**	**1.95**
Floor	**400**	Skin Limiting Core Limiting	60% 40%	55% 45%	35% 65%	11.7% 88.3%
		Fail Pass	43.3% 56.7%	45.5% 55.0%	45.5% 55.0%	48.3% 51.7%
	510	Skin Limiting Core Limiting	65% 35%	55% 45%	43% 46.7%	15% 85%
		Fail Pass	40.0% 60.0%	35.0% 65.0%	35.0% 65.0%	38.3% 61.7%
	550	Skin Limiting Core Limiting	71.3% 28.3%	66.7% 33.3%	55% 45%	30% 70%
		Fail Pass	40.0% 60.0%	35.0% 65.0%	23.3% 76.7%	15.0% 85.0%
	580	Skin Limiting Core Limiting	71.7% 28.3%	66.7% 33.3%	55% 45%	26.7% 73.3%
		Fail Pass	40.0% 60.0%	35.0% 65.0%	23.3% 76.7%	15.0% 85.0%

Table 8.9 - Span Table - Floor - 510kg/m³ - 0.55mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm			Limiting	Deflection (D)			55.0	%
Panel Width	600	mm				Shear (C)			45.0	%
Core Density	510	kg/m³				Bending (S)			0.0	%
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	4.5	DC	DC	DC	DC	DCS	DCS	DCS
100mm	12.2	9.1	7.3	4.9	D	DC	DC	DC	DCS	DCS
125mm	15.2	11.4	9.1	7.6	5.0	3.4	DC	DC	DC	DCS
150mm	18.3	13.7	11.0	9.1	7.8	6.0	4.2	DC	DC	DC
175mm	21.3	16.0	12.8	10.7	9.1	8.0	6.1	4.5	3.4	DC
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	6.2	4.6	3.6

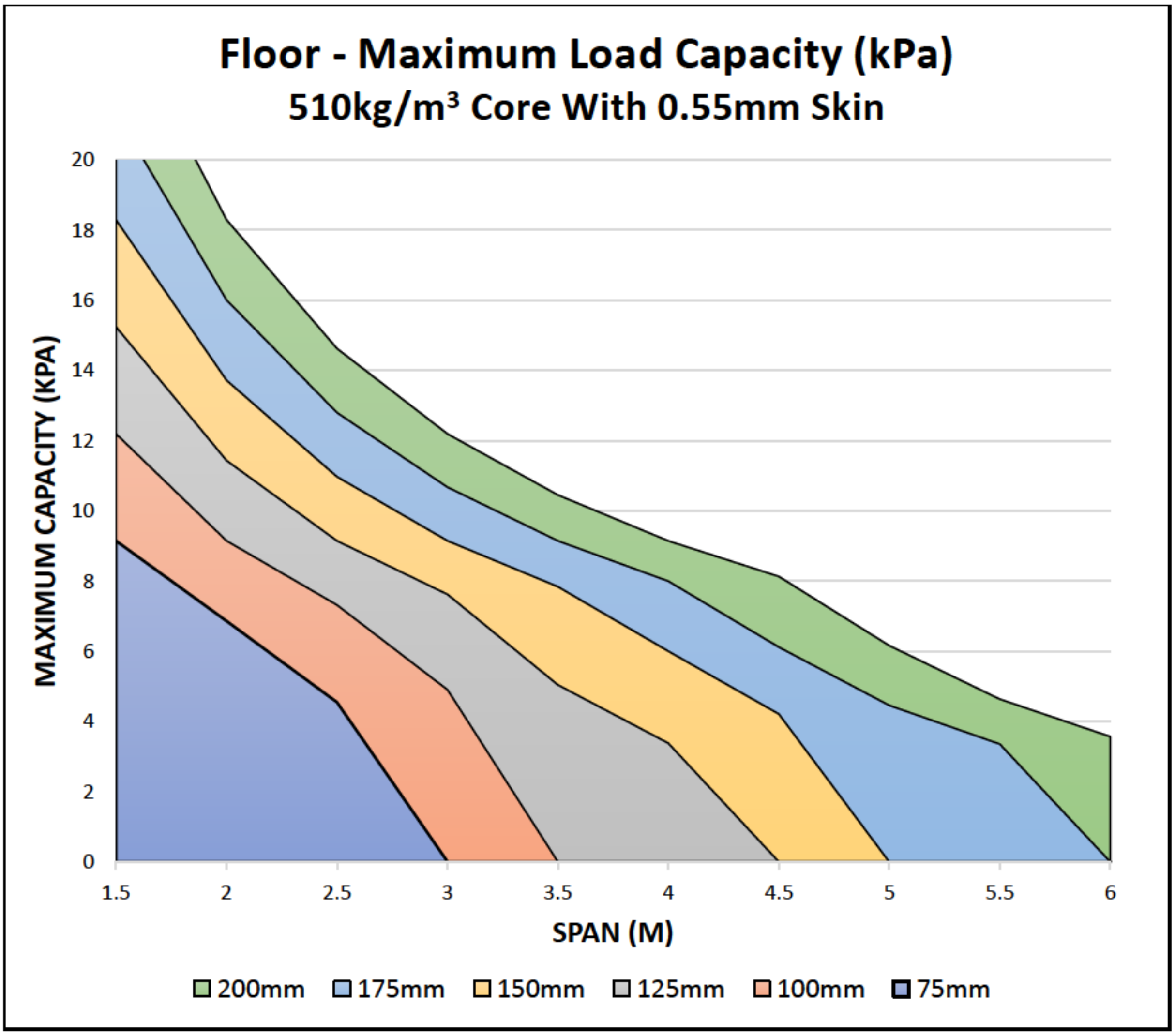

Figure 8.6 - Load Capacity Graph - Floor - 510kg/m3 - 0.55mm

Table 8.10 - Span Table - Floor - 550kg/m³ - 0.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm			Limiting	Deflection (D)		55.0	%	
Panel Width	600	mm				Shear (C)		45.0	%	
Core Density	550	kg/m³				Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.5	4.3	D	DC	DC	DC	DC	DCS
100mm	17.4	13.1	10.5	7.9	5.0	3.3	D	DC	DC	DC
125mm	21.8	16.3	13.1	10.9	8.0	5.4	3.8	D	D	DC
150mm	26.1	19.6	15.7	13.1	11.2	8.9	6.2	4.6	3.4	D
175mm	30.5	22.9	18.3	15.2	13.1	11.4	8.9	6.5	4.9	3.8
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	8.8	6.6	5.1

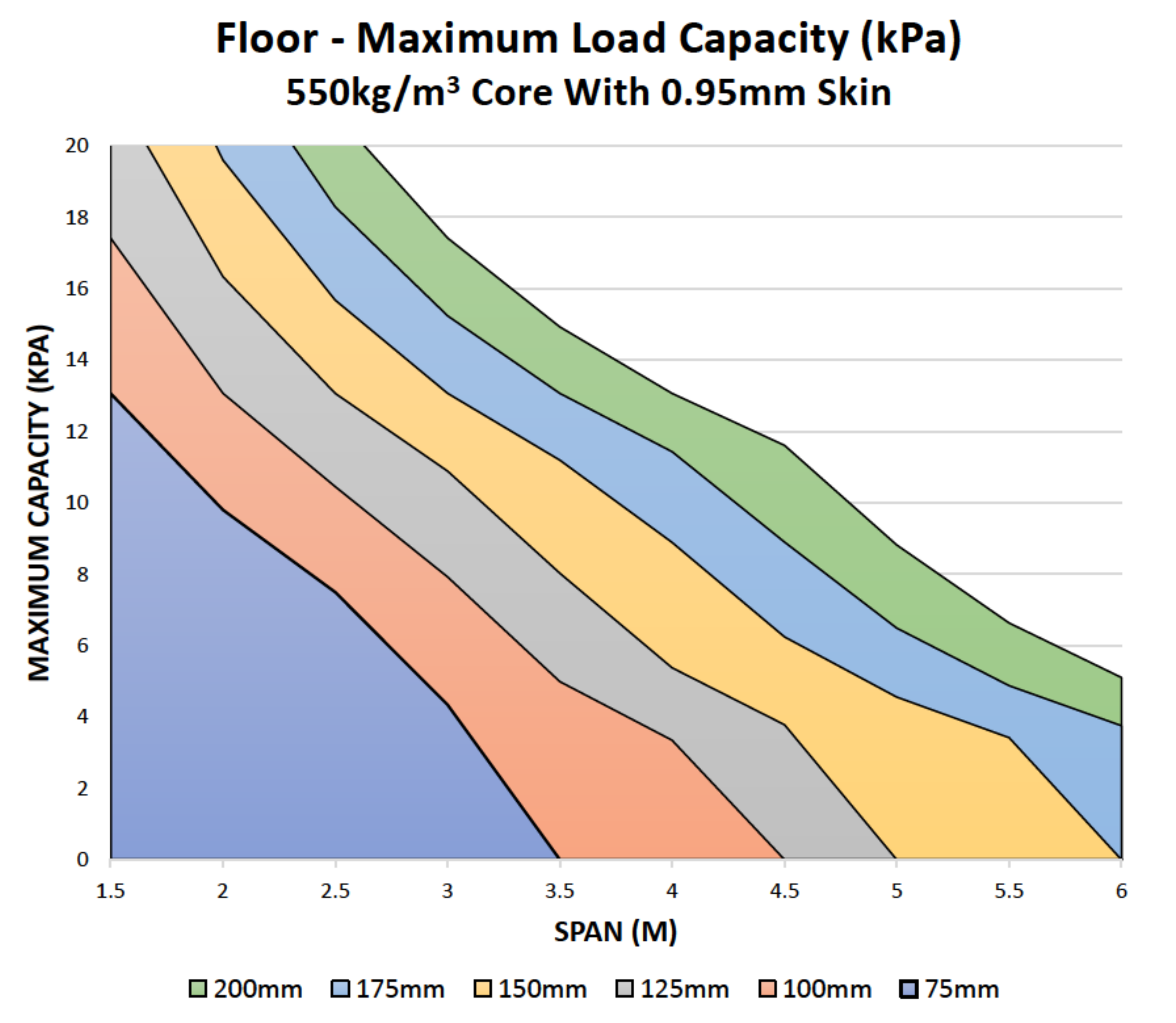

Figure 8.7 - Load Capacity Graph - Floor - 550kg/m3 - 0.95mm

8.8 Formwork

For the final investigated building application, a permanent formwork system, the decision making process chart shown in Figure 7.8 was followed to generate spans tables and respective graphical representations as presented in Appendix N. This entailed checking the capabilities of each panel against the AS3610 Formwork for Concrete requirements and the associated construction loads, followed by an analysis of the entire system (including the topping slab) in terms of performance as a floor system.

It was found that the system was able to cope with the construction loads from a stiffness perspective, though was restricted by the shear performance of the core under the significant loading that is experienced during stage 1 (prior to placement of concrete) of the formwork construction cycle. The combination of the factored 4.0kPa load for stacked materials, along with the additional live loads in combination with the dead load of the formwork system itself result in a significant total load. A summary of the overall pass/fail percentages of each span table is provided in Table 8.11 which generally shows that as a formwork system, the stressed-skin AAC core panel is inadequate for a much higher percentage than the previously investigated building applications.

The following makeup of AAC core density and steel skin thicknesses were found to be the recommended arrangements for a formwork application:

550kg/m^3 AAC core with 0.40mm steel skin (Refer to Table 8.12 and Figure 8.8)

Other findings from the formwork analysis include:

- The shear performance of the AAC core is the largest contributing factor and the cause of most failures by a significant margin.
- The skin thickness contributes to the stiffness of the formwork system, though once the topping slab is poured and contributing to create a composite floor, it becomes less critical.
- Bending failures in the form of compressive failure of the topping slab and tensile failure of the bottom sheet was not seen to occur, suggesting that as a floor system in isolation, it is effective.

Table 8.11 - Formwork Limiting Results Summary

Limiting Component Summary						
Application	**Core Density (kg/m^3)**	**Failure**	**Skin Thickness (mm)**			
			0.40	**0.55**	**0.95**	**1.95**
Formwork	**400**	Fail Pass	86.7% 13.3%	86.7% 13.3%	86.7% 13.3%	86.7% 13.3%
	510	Fail Pass	80.0% 20.0%	80.0% 20.0%	80.0% 20.0%	80.0% 20.0%
	550	Fail Pass	65.0% 35.0%	65.0% 35.0%	65.0% 35.0%	65.0% 35.0%
	580	Fail Pass	65.0% 35.0%	65.0% 35.0%	65.0% 35.0%	65.0% 35.0%

Table 8.12 - Span Table - Formwork - 550kg/m3 - 0.40mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.4 mm Panel Width 600 mm Core Density 550 kg/m^3										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	CF	CF	CF	CF	CFD	CFD
100mm	17.4	13.1	C	C	C	C	CF	CF	CF	CF
125mm	21.8	16.3	13.1	C	C	C	C	CF	CF	CF
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	CF
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

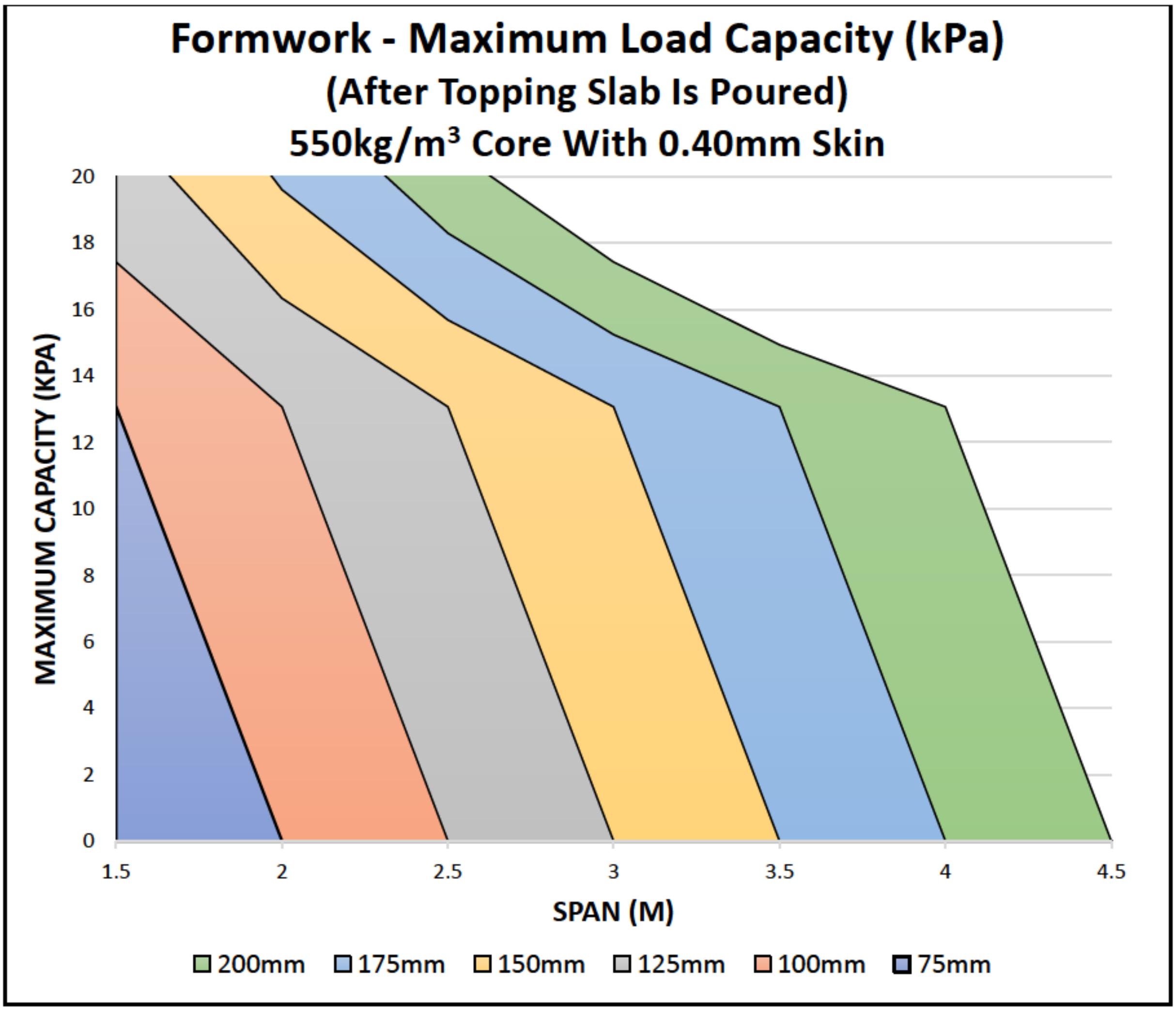

Figure 8.8 - Load Capacity Graph - Formwork - 550kg/m³ - 0.40mm

8.9 Cost

A cost estimate was developed based on prices given by the AAC and steel manufacturer as summarised in Table 8.13 (excluding the cost of the bonding material).

Table 8.13 - Cost Summary

Cost Summary ($/Panel)											
Core Thickness (mm)	Skin Thickness (mm)	Span Length (m)									
		1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75	0.40	51.4	68.6	85.7	102.9	120.0	137.2	154.3	171.5	188.6	205.8
	0.55	53.0	70.6	88.3	105.9	123.6	141.2	158.9	176.5	194.2	211.8
	0.95	57.0	76.1	95.1	114.1	133.1	152.1	171.1	190.1	209.1	228.2
	1.95	69.4	92.5	115.7	138.8	161.9	185.1	208.2	231.3	254.5	277.6
100	0.40	94.6	126.2	157.7	189.3	220.8	252.4	283.9	315.5	347.0	378.6
	0.55	96.2	128.2	160.3	192.3	224.4	256.4	288.5	320.5	352.6	384.6
	0.95	100.2	133.7	167.1	200.5	233.9	267.3	300.7	334.1	367.5	401.0
	1.95	112.6	150.1	187.7	225.2	262.7	300.3	337.8	375.3	412.9	450.4
125	0.40	106.3	141.8	177.2	212.7	248.1	283.6	319.0	354.5	389.9	425.4
	0.55	107.9	143.8	179.8	215.7	251.7	287.6	323.6	359.5	395.5	431.4
	0.95	111.9	149.3	186.6	223.9	261.2	298.5	335.8	373.1	410.4	447.8
	1.95	124.3	165.7	207.2	248.6	290.0	331.5	372.9	414.3	455.8	497.2
150	0.40	114.4	152.6	190.7	228.9	267.0	305.2	343.3	381.5	419.6	457.8
	0.55	116.0	154.6	193.3	231.9	270.6	309.2	347.9	386.5	425.2	463.8
	0.95	120.0	160.1	200.1	240.1	280.1	320.1	360.1	400.1	440.1	480.2
	1.95	132.4	176.5	220.7	264.8	308.9	353.1	397.2	441.3	485.5	529.6
175	0.40	132.4	176.6	220.7	264.9	309.0	353.2	397.3	441.5	485.6	529.8
	0.55	134.0	178.6	223.3	267.9	312.6	357.2	401.9	446.5	491.2	535.8
	0.95	138.0	184.1	230.1	276.1	322.1	368.1	414.1	460.1	506.1	552.2
	1.95	150.4	200.5	250.7	300.8	350.9	401.1	451.2	501.3	551.5	601.6
200	0.40	160.3	213.8	267.2	320.7	374.1	427.6	481.0	534.5	587.9	641.4
	0.55	161.9	215.8	269.8	323.7	377.7	431.6	485.6	539.5	593.5	647.4
	0.95	165.9	221.3	276.6	331.9	387.2	442.5	497.8	553.1	608.4	663.8
	1.95	178.3	237.7	297.2	356.6	416.0	475.5	534.9	594.3	653.8	713.2

9 Conclusions and Recommendations

This study has undertaken a flexural investigation and design of stressed-skin panels with an autoclaved aerated concrete (AAC) core, with consideration given to the use of different materials for the stressed-skin with the aim of determining its potential applications in the Australian building industry. The importance of this topic was apparent throughout the evidence in the literature review and industry perception and that some existing sandwich building systems have the potential to be hazardous from a fire safety and certification perspective.

The literature review also revealed that stressed-skin panels are commonly used effectively as floor, walls and roofing systems, though some of the foam core materials present safety concerns in the form of poor fire performance characteristics mixed with hazardous toxicity levels, which is known to be the largest single contributor to the number of deaths caused by unwanted fires.

AAC was presented as an alternative material in an attempt to address the underlying issues, with the literature review detailing its production sequence and finding that the structural characteristics and properties are effective for use in stressed-skin panels and overcome some of the inhibiting characteristics of existing core materials.

During the early stages of the project, four-point load testing was completed in order to gain further understanding as to how Hebel AAC panels perform and commonly fail, with four test samples in total:

- Test Sample 1 – Control 75mm AAC Panel (No Skins)
- Test Sample 2 – 75mm AAC Panel GFRP Skin (Faces Only)
- Test Sample 3 – 75mm AAC Panel Steel Skin (Faces Only)
- Test Sample 4 – 150mm AAC Steel Skin (Face and Sides)

The data retrieved from each test allowed an ensuing comparison and analysis against the theoretically derived predictions for each test sample, specifically for the deflection and ultimate failures. It was seen that poor bonding between the skin and core in test sample 3 resulted in a discrepancy between the test data and the calculated deflection. The improved gluing process used for test sample 4 showed good alignment between the test data and predicted deflection. The testing also revealed that the shear forces were being carried via the AAC core material (which failed at a lower applied load than expected based on the literature review findings), whilst the bending was resisted by the sandwich arrangement of the skin materials. This knowledge was used to shape the predictive analysis in the later chapters, in terms of the critical checks and expected failure modes.

With the generally positive test results and prior learning for the literature review, four potential building applications were established for investigation; wall, roof, floor and permanent formwork. Information was then extracted from the relevant Australian Standards and other codes for each respective target application, in order to develop failure criteria for each application. The central focus for the criteria was the serviceability and ultimate limit states whilst mention of fire resistance was made to show the requirements.

Using the outcomes of the testing and the obtained performance criteria, the analytical methods first presented in the testing analysis were adjusted to better reflect more realistic uniformly distributed loading conditions. Decision-making process charts were developed for each application and then applied to output span tables with associated graphs to help display the derived data. Using the data from all of the span distances, skin thickness, core thickness and core densities, idealised arrangements were determined for each application, with the efficiency of each material considered in achieving a valid solution. The data for each application highlighted the difference in loading conditions and code requirements, which led to differences in the recommended solutions for each application, which were found to be:

- Wall
    - 510kg/m^3 AAC core with 0.40mm steel skin
    - 550kg/m^3 AAC core with 0.55mm steel skin
- Roof
    - 400kg/m^3 AAC core with 0.40mm steel skin
    - 510kg/m^3 AAC core with 0.55mm steel skin
    - 550kg/m^3 AAC core with 0.95mm steel skin
- Floor
    - 510kg/m^3 AAC core with 0.55mm steel skin
    - 550kg/m^3 AAC core with 0.95mm steel skin
- Formwork
    - 550kg/m^3 AAC core with 0.40mm steel skin

The span tables display the capabilities that each configuration has, and overall the concept successfully satisfies the criteria identified in this study for each application. This indicates that the concept has the foundations to be further developed for the Australian building market in the applications presented in this dissertation to benefit all members of society including builders, developers, certifiers, insurers, engineers and the general public. If successful, it also has the capability to save lives as it will provide a safer product for the Australian building industry, whilst restoring the consumer's faith in efficient composite systems such as stressed-skin panels, which as a concept has so many overall beneficial performance characteristics to offer.

9.1 Further Work

The investigation undertaken in this report cannot capture all facets associated with the topic. As a result, there are some items which would ideally require further review and understanding to ensure the validity of this dissertation and develop the full potential of the concept. Some of the items have been highlighted throughout the earlier chapters in this report and will be recapped here, along with some additional remarks to provide areas of focus for potential future works on this topic and give the reader some insight to the potential limitations of this particular study.

- It would be beneficial to undertake additional testing to gather more data for the ultimate failure and behaviour of the combined materials to further validate the work completed in this dissertation.
- As a literature review found, the shear characteristics and capacity are somewhat unknown and vague, and the test results highlighted this as a discrepancy. Further testing could be done in this area to increase the accuracy of the analysis for the future. This would particularly assist in confirming the capabilities of the formwork system, which was most significantly limited by the shear characteristics.
- Further research and refinement in regard to the bonding of the steel skins and the AAC core should be undertaken to optimise the performance of the system, to ensure longevity is achieved and understand the influence it has on fire performance.
- Understanding the effect of services penetrations through the system from both a performance and fire rating perspective.
- Development of structural connections for use in both horizontal and vertical applications would be required to ensure that on site works is not unnecessarily difficult and to check potential crushing and bearing failures.
- A detailed cost analysis which considers all of the factors including refined material, labour and other associated building costs would help solidify the benefits of the system for the building.
- As the proposed system uses a steel sheeting as a facing material, it allows adaptation to the different roofing and wall applications which means that it has potential to be customized as required, which can serve both a visual and structural purpose. Although a ribbed profile has not been specifically addressed in this report, it can easily be used to allow the desired

aesthetic cladding finish, which can save the contractor having to install further cladding on site. Further analysis and testing would be beneficial to look at this item in regard to how it performs structurally and if this addition affects the bonding in a negative way. If successful, the system has the ability to incorporate an overlapping profile in both directions, top and bottom where necessary to ensure waterproofing is achieved to exposed surfaces and also allows fast installation to be achieved. A visual depiction of some a potential roof sheeting finish can be seen in Figure 9.1 and Figure 9.2.

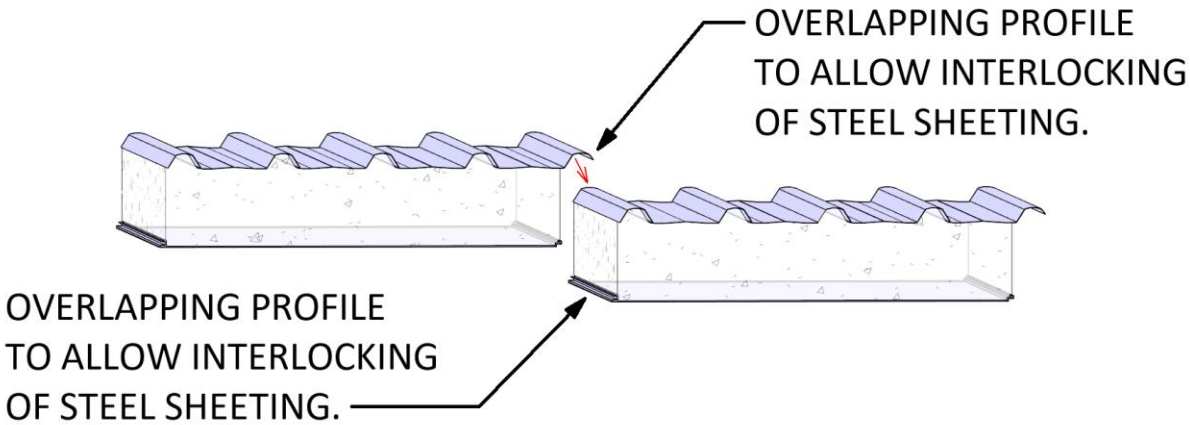

Figure 9.1 - Steel Sheeting Profile View 1

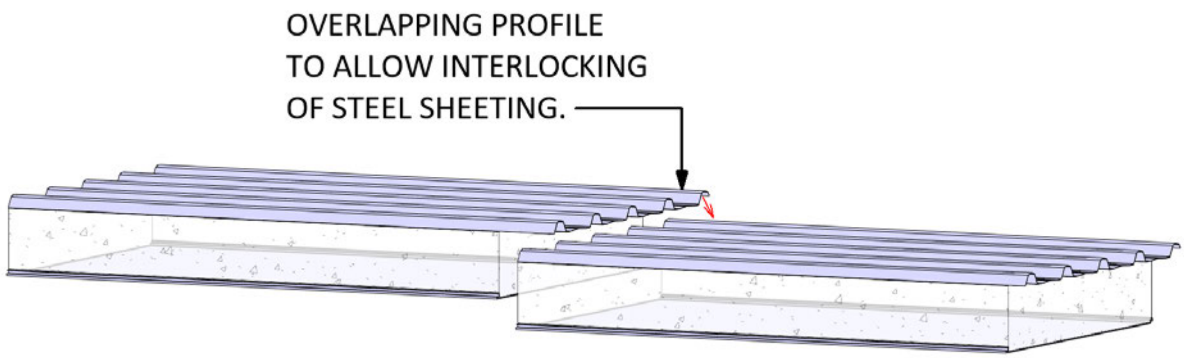

Figure 9.2 - Steel Sheeting Profile View 2

By addressing the items discussed here that required further research, testing and analysis, more positive findings could mean that the concept of a stressed-skin autoclaved aerated concrete core panel has the potential to be fully developed and implemented in the Australian building industry in the applications identified in this dissertation.

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A Appendix A

ENG4111/4112 Research Project

Project Specification

For: Brendan Shane Matthews

Title: Flexural investigation and design of stressed-skin panels with an autoclaved aerated concrete (AAC) core.

Major: Civil Engineering

Supervisor: Gary Elks

Enrolment: ENG4111 (Research Project Part 1) Semester 1 2019 External

ENG4112 (Research Project Part 2) Semester 2 2019 External

Project Aim: This study is a first step to investigate and design of stressed-skin panels with an autoclaved aerated concrete (AAC) core, and with different material systems for the stressed-skin to determine its potential applications in the Australian building industry.

Programme: Version 2, 12th October 2019

1. Review the existing stressed-skin panels and their applications that currently exist in the building industry.
2. Review the safety and certification issues with the existing stressed-skin panels systems.
3. Review autoclaved aerated concrete (AAC) and outline its characteristics.
4. Propose an area of focus/potential applications of stressed-skin panels with an autoclaved aerated concrete (AAC) core.
5. Review the Australian Standards and other literature to outline the loading conditions that the proposed system will encounter, in order to establish the performance criteria.
6. Design the stressed-skin panels with an autoclaved aerated concrete (AAC) core with differing skin and core configurations.
7. Provide the 'idealised' specifications of the design outcomes, giving the best core configuration and skin material in the form of a table with various spans.

If time and resources permit:

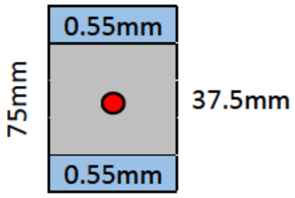
8. Physically test various configurations to validate the expectations and findings.
9. Undertake a cost analysis of the system in comparison to the existing methods of construction in the identified applications.

B Appendix B

The flexural rigidity (EI) extended calculations for test samples 3 and 4 discussed in section of this report 5.2.1 are given below:

Test Sample 3

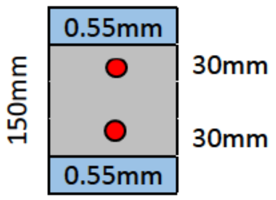
Ø of bars (mm)	4
No. Bars	3
No. of layers	1



Area A (mm^2)	Depth d (mm)	Ad^2	I	I (total)	EI ($kN.m^2$)	
330	37.775	470893.7	8.3	470902	94.1804	Steel Skin
37.70	0	0	12.6	12.6	0.0025	Reo Layer 1
45000	0	0	21093750.0	21093750	33.6445	Core
N/A	N/A	N/A	N/A	N/A	N/A	Reo Layer 2
330	37.775	470893.7	8.3	470902	94.1804	Steel Skin
					222.01	Total EI ($kN.m^2$)

Test Sample 4

Ø of bars (mm)	6
No. Bars	8
No. of layers	2

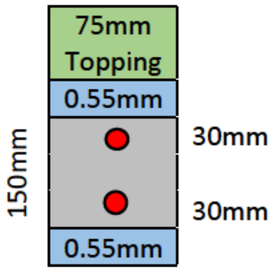


Area A (mm^2)	Depth d (mm)	Ad^2	I	I (total)	EI ($kN.m^2$)	
330	75.275	1869887	8.3	1869895.8	373.9792	Steel Skin
226.19	45	458044	63.6	458107.8	91.6216	Reo Layer 1
90000	0	0	168750000	168750000	269.1563	Core
226.19	45	458044	63.6	458107.8	91.6216	Reo Layer 2
330	75.275	1869887	8.3	1869895.8	373.9792	Steel Skin
					1200.36	Total EI ($kN.m^2$)

C Appendix C

An example calculation for the flexural rigidity (EI) extended calculations for are given below which gives consideration for a 75mm topping slab, followed by an example calculation for finding the neutral axis. This specifications for this example has a core density of 510kg/m3, a core thickness of 150mm and a steel skin thickness of 0.55mm. This matches the conditions used in test sample 4 with the addition of the topping slab.

Ø of bars (mm)	6
No. Bars	8
No. of layers	2



Area A (mm^2)	Depth d (mm)	Ad^2	I	I (total)	EI ($kN.m^2$)	
45000	26.398	31358220.61	21093750	52451970.6	1400.5	Topping
330	11.377	42714.6418	8.3	42723.0	8.5446	Steel Skin
226.19	41.652	392424.4751	63.6	392488.1	78.4976	Reo Layer 1
90000.00	86.652	675772713.4	168750000.0	844522713.4	1347.0137	Core
226.19	131.652	3920468.09	63.6	3920531.7	784.1063	Reo Layer 2
330	161.927	8652726.835	8.3	8652735.2	1730.5470	Steel Skin
					5349.18	Total EI (kN.m2)

AE	Distance of Centre From Bottom Layer (D)	AE x D	
1201500000	188.6	2.26603E+11	Topping
66000000	150.825	9954450000	Steel Skin
45238934.21	120.55	5453553519	Reo Layer 1
143550000	75.55	10845202500	Core
45238934.21	30.55	1382049440	Reo Layer 2
66000000	0.275	18150000	Steel Skin
1567527868		2.54256E+11	Sum
	Centroid (Distance From Bottom)	**162.20**	mm

D Appendix D

Table D.1 - Classification of buildings covered in BCA

Classes of Building		
Class 1	Class 1a	A single dwelling being a detached house, or one or more attached dwellings, each being a building, separated by a fire-resisting wall, including a row house, terrace house, town house or villa unit.
	Class 1b	A boarding house, guest house, hostel or the like with a total area of all floors not exceeding 300m^2, and where not more than 12 reside, and is not located above or below another dwelling or another Class of building other than a private garage.
Class 2	A building containing 2 or more sole-occupancy units each being a separate dwelling.	
Class 3	A residential building, other than a Class 1 or 2 building, which is a common place of long term or transient living for a number of unrelated persons. Example: boarding-house, hostel, backpackers accommodation or residential part of a hotel, motel, school or detention centre.	
Class 4	A dwelling in a building that is Class 5, 6, 7, 8 or 9 if it is the only dwelling in the building.	
Class 5	An office building used for professional or commercial purposes, excluding buildings of Class 6, 7, 8 or 9.	
Class 6	A shop or other building for the sale of goods by retail or the supply of services direct to the public. Example: café, restaurant, kiosk, hairdressers, showroom or service station.	
Class 7	Class 7a	A building which is a car park.
	Class 7b	A building which is for storage or display of goods or produce for sale by wholesale.
Class 8	A laboratory, or a building in which a handicraft or process for the production, assembling, altering, repairing, packing, finishing or cleaning of goods or produce is carried on for trade, sale or gain.	
Class 9	A building of a public nature.	
	Class 9a	A health care building, including those parts of the building set aside as a laboratory.
	Class 9b	An assembly building, including a trade workshop, laboratory or the like, in a primary or secondary school, but excluding any other parts of the building that are of another class.
	Class 9c	An aged care building.

Class 10	A non-habitable building or structure.	
	Class 10a	A private garage, carport, shed or the like.
	Class 10b	A structure being a fence, mast, antenna, retaining or free standing wall, swimming pool or the like.
	Class 10c	A private bushfire shelter.

(ABCB 2019)

E Appendix E

Table E.1 - Type of construction for buildings covered in BCA

Type of Construction		
Rise in storeys	**Class of building 2, 3, 9**	**Class of building 5, 6, 7, 8**
4 or more	A	A
3	A	B
2	B	C
1	C	C

(ABCB 2019)

F Appendix F

Table F.1 - BCA Volume 1 Type A construction: FRL of building elements

Building Element	**Class of building — FRL: (in minutes) Structural adequacy/Integrity/Insulation**			
	2, 3 or 4 part	**5, 7a or 9**	**6**	**7b or 8**
EXTERNAL WALL (including any column and other building element incorporated within it) or other external building element, where the distance from any fire-source feature to which it is exposed is—				
For loadbearing parts—				
less than 1.5 m	90/ 90/ 90	120/120/120	180/180/180	240/240/240
1.5 to less than 3 m	90/ 60/ 60	120/ 90/ 90	180/180/120	240/240/180
3 m or more	90/ 60/ 30	120/ 60/ 30	180/120/ 90	240/180/ 90
For non- loadbearing parts—				
less than 1.5 m	–/ 90/ 90	–/120/120	–/180/180	–/240/240
1.5 to less than 3 m	–/ 60/ 60	–/ 90/ 90	–/180/120	–/240/180
3 m or more	–/–/–	–/–/–	–/–/–	–/–/–
EXTERNAL COLUMN not incorporated in an external wall—				
For loadbearing columns—	90/–/–	120/–/–	180/–/–	240/–/–
For non-loadbearing columns—	–/–/–	–/–/–	–/–/–	–/–/–
COMMON WALLS and FIRE WALLS—	90/ 90/ 90	120/120/120	180/180/180	240/240/240
INTERNAL WALLS—				
Fire-resisting lift and stair shafts—				
Loadbearing	90/ 90/ 90	120/120/120	180/120/120	240/120/120
Non- loadbearing	–/ 90/ 90	–/120/120	–/120/120	–/120/120
Bounding public corridors, public lobbies and the like—				
Loadbearing	90/ 90/ 90	120/–/–	180/–/–	240/–/–

Non- loadbearing	–/ 60/ 60	–/–/–	–/–/–	–/–/–
Between or bounding sole-occupancy units—				
Loadbearing	90/ 90/ 90	120/–/–	180/–/–	240/–/–
Non- loadbearing	–/ 60/ 60	–/–/–	–/–/–	–/–/–
Ventilating, pipe, garbage, and like shafts not used for the discharge of hot products of combustion—				
Loadbearing	90/ 90/ 90	120/ 90/ 90	180/120/120	240/120/120
Non- loadbearing	–/ 90/ 90	–/ 90/ 90	–/120/120	–/120/120
OTHER LOADBEARING INTERNAL WALLS, INTERNAL BEAMS, TRUSSES				
and COLUMNS—	90/–/–	120/–/–	180/–/–	240/–/–
FLOORS	90/ 90/ 90	120/120/120	180/180/180	240/240/240
ROOFS	90/ 60/ 30	120/ 60/ 30	180/ 60/ 30	240/ 90/ 60

(ABCB 2019)

G Appendix G

Table G.1 - Suggested Serviceability Limit State Criteria Table C1 AS1170.0

Element	Phenomenon Controlled	Serviceability Parameter	Applied Action	Element Response (see Notes 1 and 2 AS1170.0)
Roof cladding Metal roof cladding Concrete or ceramic roof cladding	 Indentation De-coupling	 Residual deformation Mid-span deflection Mid-span deflection	 Q = 1 kN [G, ψ_sQ] [G, ψ_sQ]	 Span/600 but <0 5 mm Span/120 Span/400
Roof-supporting elements Roof members (trusses, rafters, etc.) Roof elements supporting brittle claddings	 Sag Cracking	 Mid-span deflection Mid-span deflection	 [G, ψ_lQ] [G, ψ_sQ] or [W_s]	 Span/300 Span/400
Ceiling and ceiling supports Ceilings with matt or gloss paint finish Ceilings with textured finish Suspended ceilings Ceiling support framing Ceilings with plaster finish	 Ripple Ripple Ripple Sag Cracking	 Mid-span deflection Mid-span deflection Mid-span deflection Mid-span deflection Mid-span deflection	 G G G G [G, ψsQ] or [Ws]	Span/500 (see Note 3) Span/300 Span/360 Span/360 Span/200
Wall elements Columns Portal frames (frame racking action) Lintel beams (vertical sag) Walls—General (face loaded)	Side sway Roof damage Doors/windows jam Discerned movement Impact: soft body (Neighbours notice)	Deflection at top Deflection at top Mid-span deflection Mid-height deflection Mid-height deflection	Ws [Ws] or [E_s] Ws Ws Q = 0.7 kN	Height/500 Spacing/200 (Note 4) Span/240 but <12 mm (see Note 5) Height/150 Height/200 but <12 mm (see Note 6)
	Supported elements rattle	Mid-height deflection	Ws	Height/1000
Walls—Specific claddings:				
Brittle cladding (ceramic) face loaded Masonry walls (in plane) (face loading) Plaster/gypsum walls (in plane) (face loading) Movable partitions (soft body impact) Glazing systems Windows, facades, curtain walls Fixed glazing systems	Cracking Noticeable cracking Noticeable cracking Lining damage Lining damage System damage Bowing Facade damage Glass damage	Mid-height deflection Deflection at top Deflection at top Mid-height deflection Mid-height deflection Deflection at top Mid-span deflection Mid-span deflection Deflection	Ws [Ws] or [E_s] [Ws] or [E_s] Ws [Ws] or [Es] Q = 0.7 kN Ws [Ws] or [Es] [Ws] or [Es]	Height/500 Height/600 Height/400 Height/300 Height/200 Height/160 Span/400 Span/250 2 × glass clearance (see Note 3)
Floors and floor supports Beams where line-of-sight is along invert Beams where line-of-sight is across soffit Flooring Floor joists/beams Floors Normal floor systems Specialist floor systems Floors—Side-sway (acceleration) Floors—Supporting masonry walls Floors—Supporting plaster lined walls Floors supporting existing masonry walls—Underpinning floors Floors—For access for working by operators and maintenance Handrails—Post and rail system	 Sag Sag Ripple Sag Vibration Noticeable sag Noticeable sag Sway Wall cracking Cracks in lining Wall cracking Sag Side sway	 Mid-span deflection Mid-span deflection Mid-span deflection Mid-span deflection Static midspan deflection Mid-span deflection Mid-span deflection Acceleration at floor Mid-span deflection Mid-span deflection Mid-span deflection Midspan deflection Mid-span system deflection	 [G, ψlQ] [G, ψlQ] [G, ψlQ] [G, ψlQ] Q = 1.0 kN [G, ψlQ] [G, ψlQ] Ws (P=5) [G, ψlQ] [G, ψlQ] [G, ψlQ] Q = 1 kN Q = 1.5 kN/m	 Span/500 (see Notes 8, 9) Span/250 Span/300 Span/300 less than 1 to 2 mm (see Note 10) Span/400 Span/600 <0 01g (see Note 11) Span/500 Span/300 Span/750 Span/250 Height/60 + Span/240

(Standard Australia 2002a)

H Appendix H

Table H.1 - Serviceability Pressures (kPa) For Wind Classification – Table 3.4 AS4055

Wind Class	**Walls**			**Roofs**			
	Any position	Away from corners	Within 1200 mm of corners	Any position	General away from edges	Within 1200 mm of edges	At corners (within 1200 mm of both edges)
Pressure Zone	G, SC	G	SC	G, RE, RC	G	RE	RC
$K_c.C_{p,n}$	+0.9	-0.77	-1.35	+0.63	-0.99	-1.8	-2.61
$N1_{serv}$	+0.37	-0.31	-0.55	+0.26	-0.40	-0.73	-1.06
$N2_{serv}$	+0.37	-0.31	-0.55	+0.26	-0.40	-0.73	-1.06
$N3_{serv}$	+0.55	-0.47	-0.83	+0.39	-0.61	-1.11	-1.60
$N4_{serv}$	+0.82	-0.70	-1.23	+0.57	-0.90	-1.64	-2.38
$N5_{serv}$	+1.19	-1.02	-1.79	+0.84	-1.31	-2.39	-3.46
$N6_{serv}$	+1.63	-1.40	-2.45	+1.14	-1.80	-3.27	-4.74
$K_c.C_{p,n}$	+0.9	-0.77	-1.35	+0.63	-0.99	-1.8	-2.61
$C1_{serv}$	+0.55	-0.47	-0.83	+0.39	-0.61	-1.11	-1.60
$C2_{serv}$	+0.82	-0.70	-1.23	+0.57	-0.90	-1.64	-2.38
$C3_{serv}$	+1.19	-1.02	-1.79	+0.84	-1.31	-2.39	-3.46
$C4_{serv}$	+1.63	-1.40	-2.45	+1.14	-1.80	-3.27	-4.74

(Standard Australia 2012)

I Appendix I

Table I.1 - Ultimate Strength Pressures (kPa) For Wind Classification – Table 3.3 AS4055

Wind Class	Walls			Roofs			
	Any position	Away from corners	Within 1200 mm of corners	Any position	General away from edges	Within 1200 mm of edges	At corners (within 1200 mm of both edges)
Pressure Zone	G, SC	G	SC	G, RE, RC	G	RE	RC
$K_c.C_{p,n}$	+0.9	-0.77	-1.35	+0.63	-0.99	-1.8	-2.61
$N1_{serv}$	+0.62	-0.53	-0.94	+0.44	-0.69	-1.25	-1.81
$N2_{serv}$	+0.86	-0.74	-1.30	+0.60	-0.95	-1.73	-2.51
$N3_{serv}$	+1.35	-1.16	-2.03	+0.98	-1.49	-2.70	-3.92
$N4_{serv}$	+2.01	-1.73	-3.01	+1.41	-2.21	-4.02	-5.83
$N5_{serv}$	+2.96	-2.53	-4.44	+2.07	-3.25	-5.91	-8.58
$N6_{serv}$	+3.99	-3.42	-5.99	+2.80	-4.39	-7.99	-11.58
$K_c.C_{p,n}$	+1.2	-1.2	-1.8	+0.95	-1.44	-2.25	-3.06
$C1_{serv}$	+1.80	-1.80	-2.7	+1.43	-2.16	-3.38	-4.59
$C2_{serv}$	+2.68	-2.68	-4.03	+2.12	-3.21	-5.02	-6.83
$C3_{serv}$	+3.94	-3.94	-5.91	+3.12	-4.73	-7.39	-10.05
$C4_{serv}$	+5.33	-5.33	-7.99	+4.22	-6.39	-9.98	-13.58

(Standard Australia 2012)

J Appendix J

Table J.1 - Table 4.5.1 AS3610

1	2	3	4	5	6	7	8	9	10	11	12	13
Equation No.	**Stage**	**Dead Loads**		**Loads of Medium Duration**					**Loads of Short Duration**			
		Formwork	**Concrete**	**Uniform live loads**	**Loads from stacked materials**	**Water (river currents, tidal, wave action)**	**Misc. loads — prestress, settlement, axial shortening, shrinkage, etc. (see Clauses 4.4.2.5 and 4.4.5.7)**	**Lateral concrete pressure**	**Concentrated live load**	**Wind (or wind and flooding)**	**Horizontal impact (see Clause 4.4.5.3(c))**	**Global load factor for primary members (see Note 8)**
		G	**G_c**	**Q_{uv} or Q_{uh}**	**M**	**X_w**	**X_m**	**P**	**Qc**	**W_u**	**I**	
1		1.25G	-	$+1.5Q_{uv}+1.5Q_{uh}$	-	$+1.5X_w$	-	-	-	-	-	1.3
2*		0.8G	-	-	-	$+1.5X_w$	-	-	-	$+W_u$	-	1
3	I	0.8G	-	-	-	-	-	-	-	$+W_u$	-	1
4†		0.8G	-	-	-		-	-	-	-	+1.1I	1
5		G	-	Q_{uv}	$+M_1$	-	-	-	-	-	+1.1I	1
6		1.25G	$+1.25G_c$	$+1.5Q_{uv}+1.5Q_{uh}$	$+1.5M_2$	$+1.5X_w$	-	+1.5P	-	-	-	1.3
7	II	1.25G	$+1.25G_c$	-	-	-	-	-	$+Q_c$	-	-	1.3
8		G	$+G_c$	$+1.1Q_{uv}$	$+1.1M_2$	-	-	-	-	-	+1.1I	1
9		1.25G	$+1.25G_c$	$+1.5Q_{uv}+1.5Q_{uh}$	$+1.5M_3$	$+1.5X_w$	$+1.5X_w$	-	-	-	-	1.3
10	III	1.25G	$+1.25G_c$	-	$+1.5M_3$	$+1.5X_w$	$+1.25X_w$	-	-	$+W_u$	-	1.3
Factored Load Combinations For Stiffness												
11	II	G	$+G_c$	-	-	-	-	-	-	-	-	1
12	II	-	-	-	-	-	-	P	-	-	-	1
13	III	G	$+G_c$	-	$+M_3$	-	-	-	-	-	-	1

(Standard Australia 1995)

This following appendices (Appendix K, Appendix L, Appendix M and Appendix N) show every span table that was generated for each application (wall, roof, floor and formwork), from which the results section of this report used to provide to recommendation on the arrangements.

K Appendix K

Table K.1 - Span Table - Wall - 400kg/m³ - 0.40mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)		40.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		60.0	%		
Core Density	400	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.7	3.1	2.0	1.3	0.9	0.7	D	D
100mm	10.5	7.8	6.3	5.2	3.7	2.4	1.7	1.3	0.9	0.7
125mm	13.1	9.8	7.8	6.5	5.6	4.0	2.8	2.1	1.5	1.2
150mm	15.7	11.8	9.4	7.8	6.7	5.9	5.2	3.9	2.9	2.2
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	4.2	3.3
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	4.5

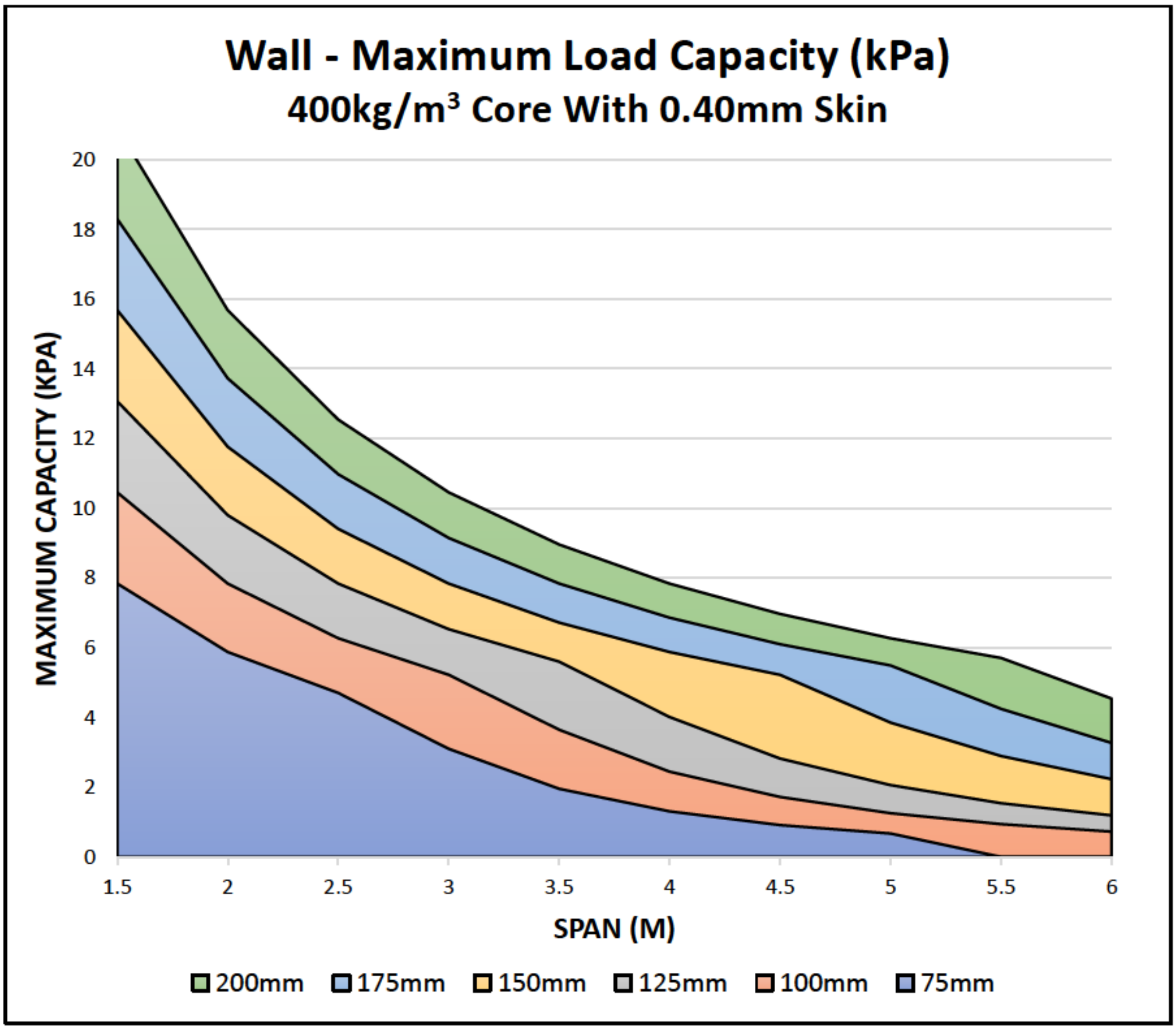


Figure K.1 - Load Capacity Graph - Wall - 400kg/m³ - 0.40mm

Table K.2 - Span Table - Wall - 400kg/m³ - 0.55mm

Wall Application Maximum Load Capacity (kPa) Table

Single Span – Limited by Serviceability & Ultimate Limit State

Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)	31.7	%
Panel Width	600	mm	N3 Region Max		Shear (C)	68.3	%
Core Density	400	kg/m³	N5 Region Max		Bending (S)	0.0	%

Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.7	3.9	2.6	1.7	1.2	0.9	0.7	D
100mm	10.5	7.8	6.3	5.2	4.5	3.2	2.2	1.6	1.2	0.9
125mm	13.1	9.8	7.8	6.5	5.6	4.9	3.6	2.6	2.0	1.5
150mm	15.7	11.8	9.4	7.8	6.7	5.9	5.2	4.7	3.5	2.7
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	3.9
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	5.2

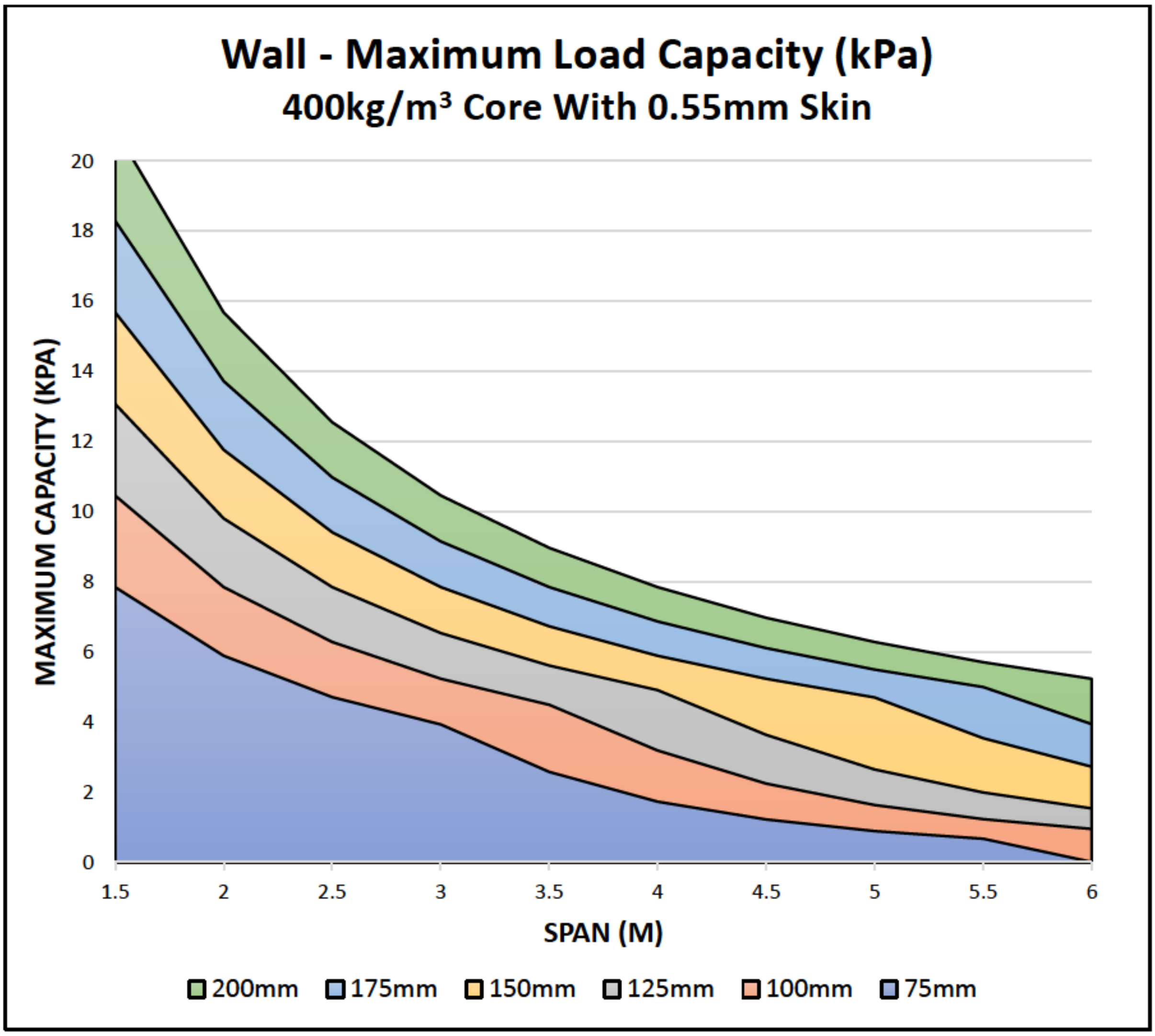


Figure K.2 - Load Capacity Graph - Wall - 400kg/m³ - 0.55mm

Table K.3 - Span Table - Wall - 400kg/m³ - 0.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max	Limiting	Deflection (D)		16.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		83.3	%		
Core Density	400	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.7	3.9	3.4	2.8	2.0	1.5	1.1	0.8
100mm	10.5	7.8	6.3	5.2	4.5	3.9	3.5	2.6	2.0	1.5
125mm	13.1	9.8	7.8	6.5	5.6	4.9	4.4	3.9	3.2	2.4
150mm	15.7	11.8	9.4	7.8	6.7	5.9	5.2	4.7	4.3	3.9
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	4.6
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	5.2

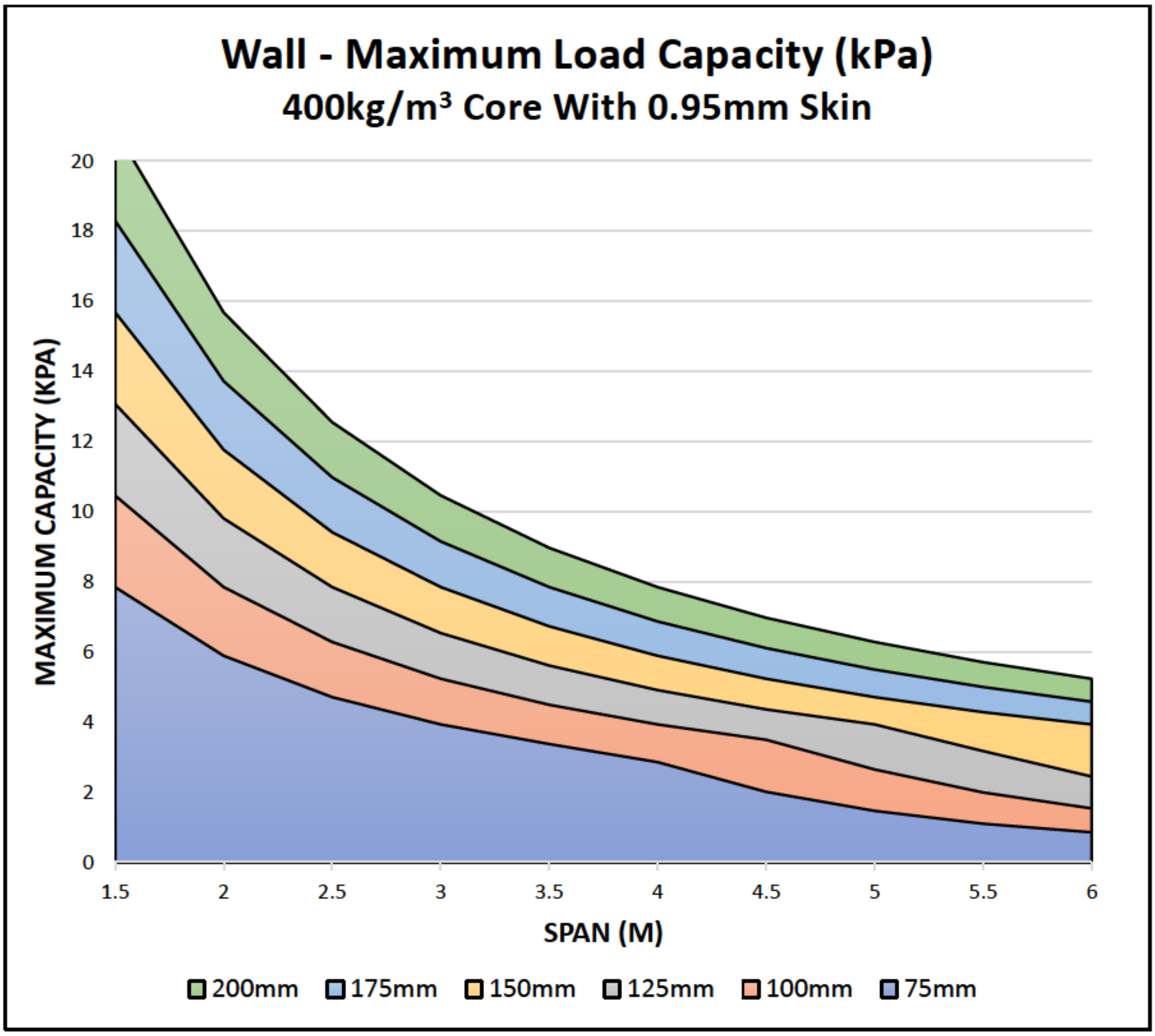


Figure K.3 - Load Capacity Graph - Wall - 400kg/m³ - 0.95mm

Table K.4 - Span Table - Wall - 400kg/m³ - 1.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)		1.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		98.3	%		
Core Density	400	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.7	3.9	3.4	2.9	2.6	2.4	2.1	1.7
100mm	10.5	7.8	6.3	5.2	4.5	3.9	3.5	3.1	2.9	2.6
125mm	13.1	9.8	7.8	6.5	5.6	4.9	4.4	3.9	3.6	3.3
150mm	15.7	11.8	9.4	7.8	6.7	5.9	5.2	4.7	4.3	3.9
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	4.6
200mm	20.0	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	5.2

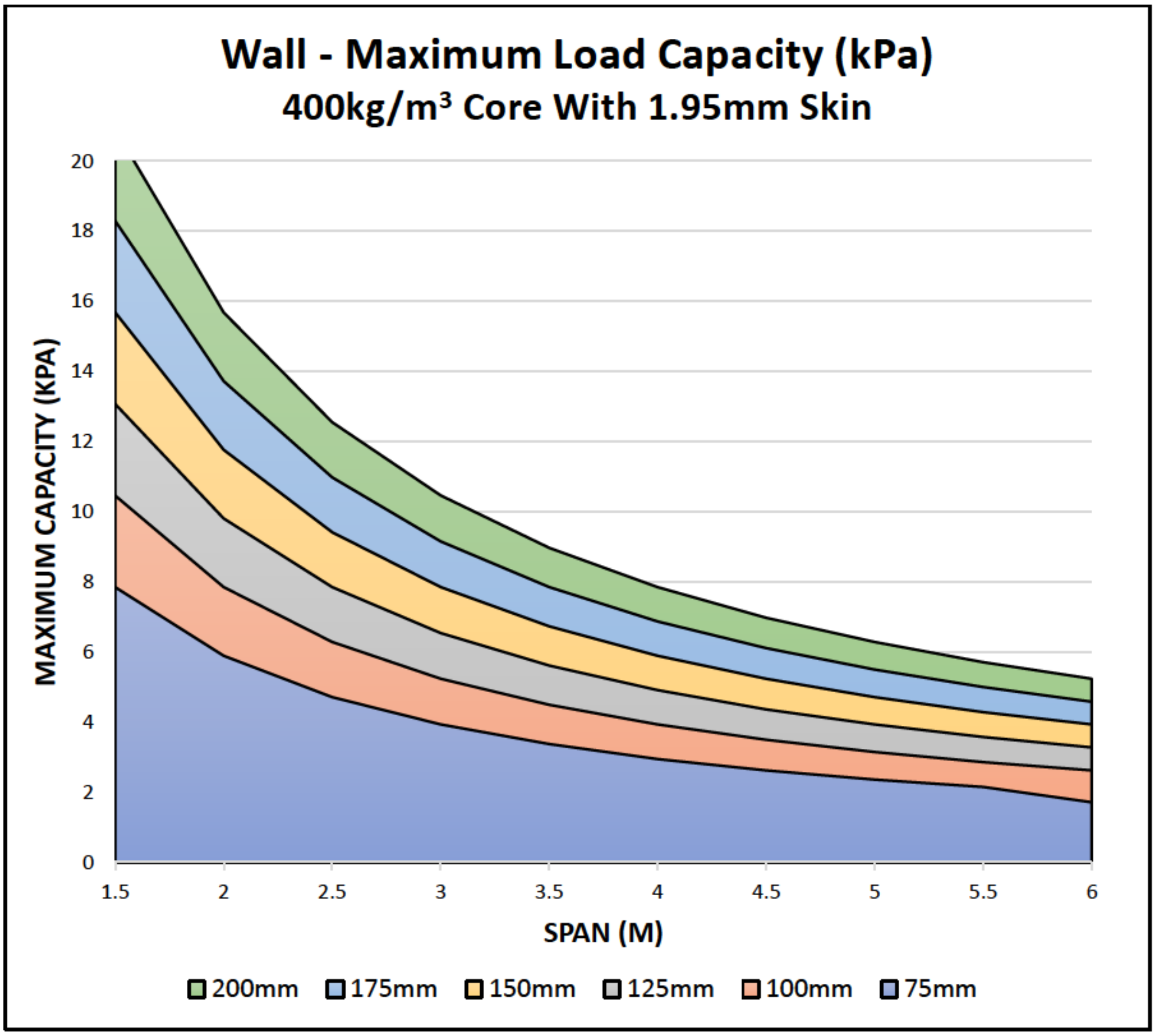


Figure K.4 - Load Capacity Graph - Wall - 400kg/m³ - 1.95mm

Table K.5 - Span Table - Wall - 510kg/m3 - 0.40mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)		46.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		53.3	%		
Core Density	510	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	3.2	2.0	1.4	1.0	0.7	D	D
100mm	12.2	9.1	7.3	6.1	3.8	2.6	1.8	1.3	1.0	0.8
125mm	15.2	11.4	9.1	7.6	6.4	4.3	3.0	2.2	1.6	1.3
150mm	18.3	13.7	11.0	9.1	7.8	6.9	5.6	4.1	3.1	2.4
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.0	4.5	3.5
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.3	4.8

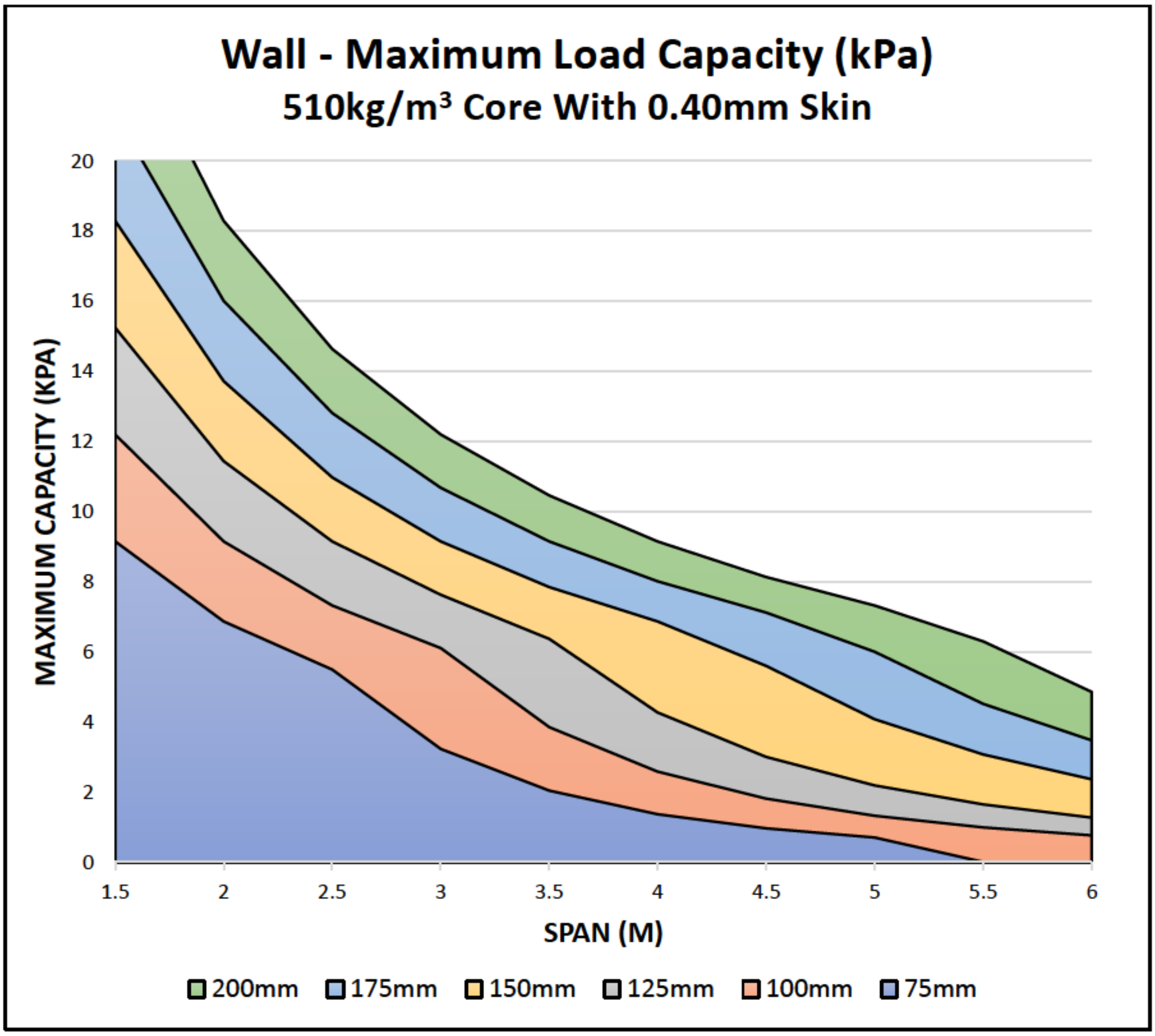


Figure K.5 - Load Capacity Graph - Wall - 510kg/m³ - 0.40mm

Table K.6 - Span Table - Wall - 510kg/m3 - 0.55mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)		40.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		60.0	%		
Core Density	510	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	4.2	2.7	1.8	1.2	0.9	0.7	D
100mm	12.2	9.1	7.3	6.1	4.9	3.3	2.3	1.7	1.3	1.0
125mm	15.2	11.4	9.1	7.6	6.5	5.4	3.8	2.8	2.1	1.6
150mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	4.9	3.7	2.8
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.4	5.4	4.1
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.7	5.7

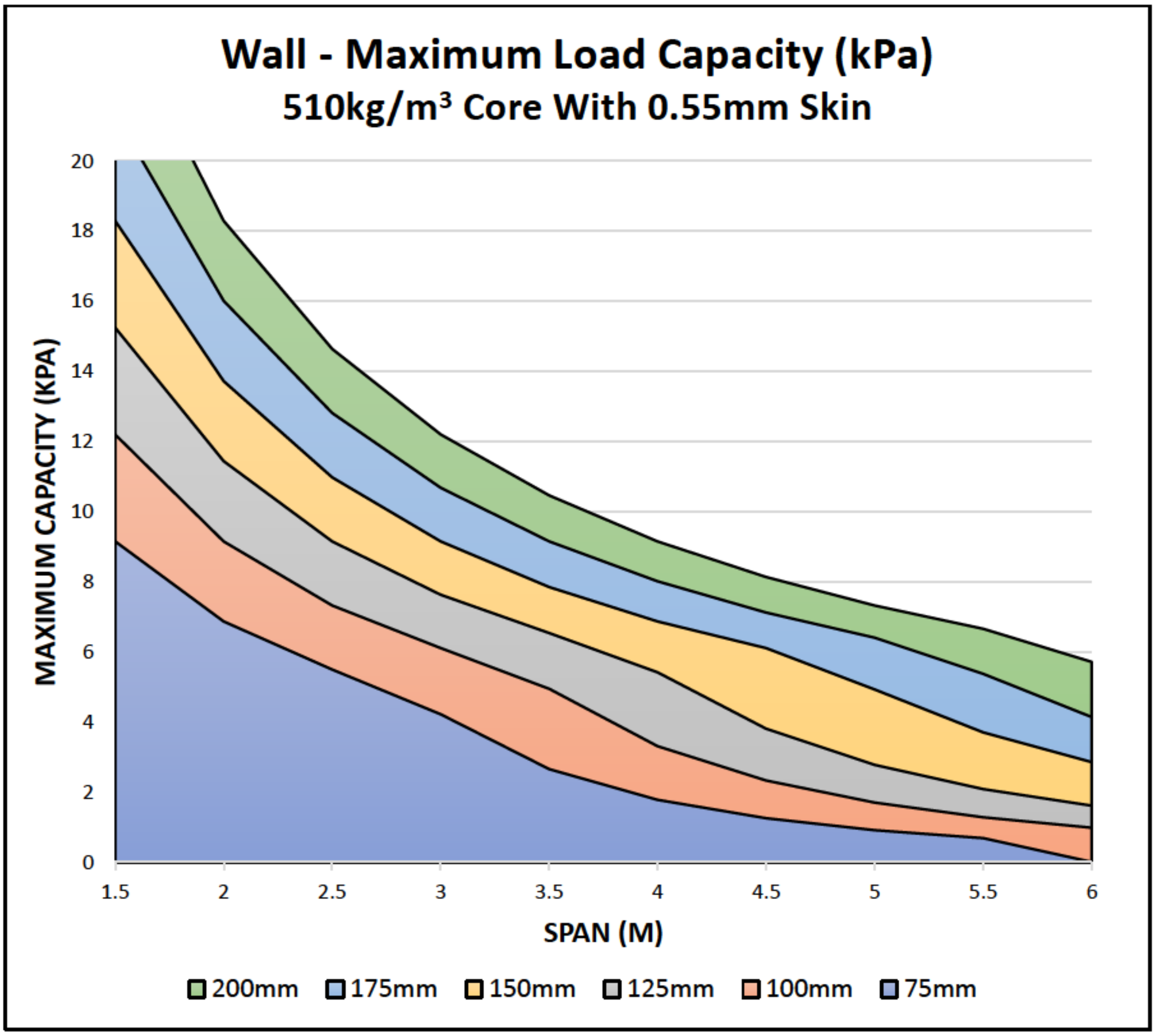


Figure K.6 - Load Capacity Graph - Wall - 510kg/m³ - 0.55mm

Table K.7 - Span Table - Wall - 510kg/m3 - 0.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.95 mm			N1 Region Max		Limiting	Deflection (D)			21.7 %	
Panel Width 600 mm			N3 Region Max			Shear (C)			78.3 %	
Core Density 510 kg/m³			N5 Region Max			Bending (S)			0.0 %	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	4.6	3.9	2.9	2.0	1.5	1.1	0.9
100mm	12.2	9.1	7.3	6.1	5.2	4.6	3.7	2.7	2.0	1.6
125mm	15.2	11.4	9.1	7.6	6.5	5.7	5.1	4.3	3.3	2.5
150mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	4.2
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.4	5.8	5.3
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.7	6.1

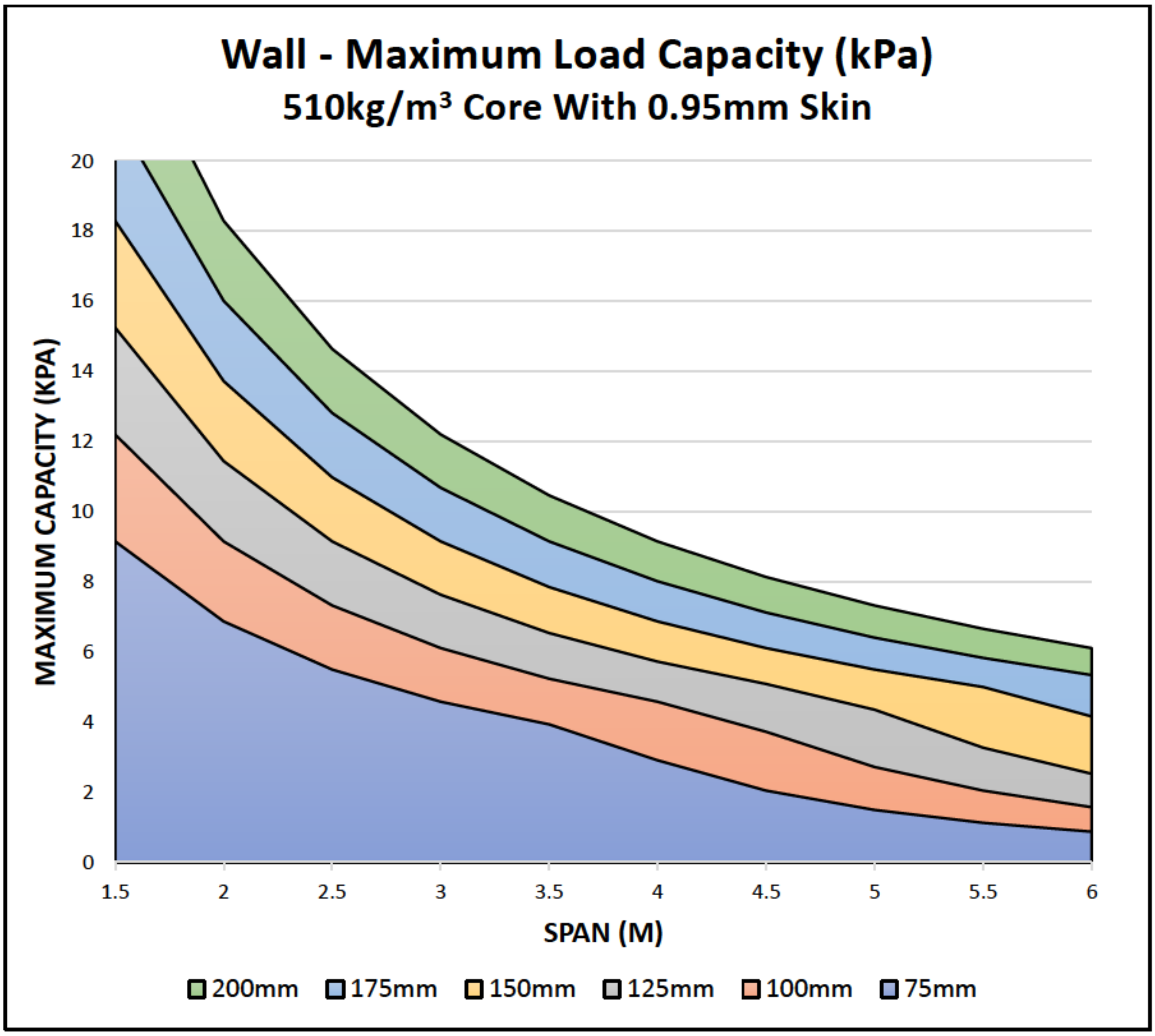


Figure K.7 - Load Capacity Graph - Wall - 510kg/m³ - 0.95mm

Table K.8 - Span Table - Wall - 510kg/m3 - 1.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)	3.3	%			
Panel Width	600	mm	N3 Region Max		Shear (C)	96.7	%			
Core Density	510	kg/m³	N5 Region Max		Bending (S)	0.0	%			
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	4.6	3.9	3.4	3.0	2.7	2.2	1.7
100mm	12.2	9.1	7.3	6.1	5.2	4.6	4.1	3.7	3.3	3.0
125mm	15.2	11.4	9.1	7.6	6.5	5.7	5.1	4.6	4.2	3.8
150mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	4.6
175mm	20.0	16.0	12.8	10.7	9.1	8.0	7.1	6.4	5.8	5.3
200mm	20.0	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.7	6.1

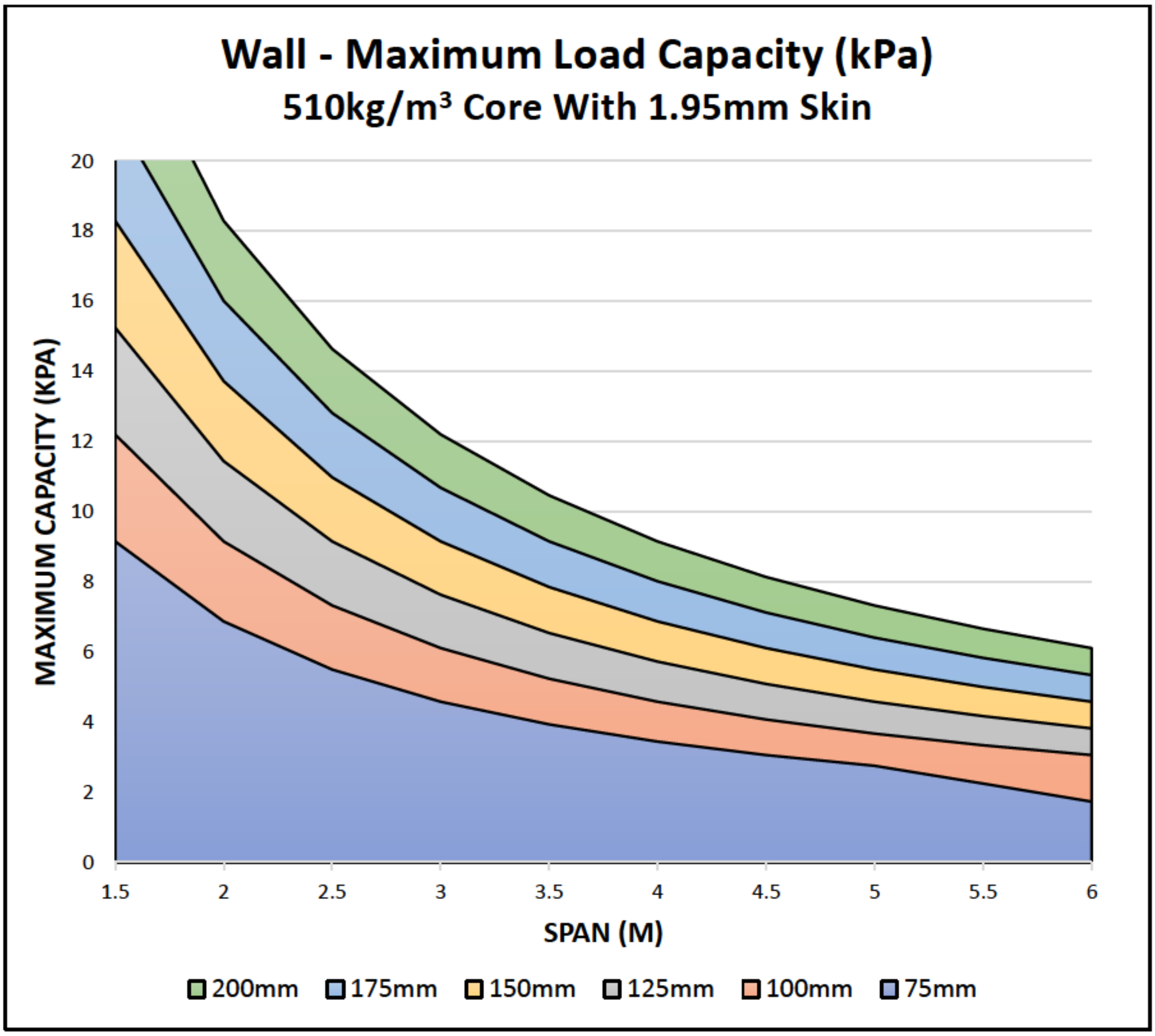


Figure K.8 - Load Capacity Graph - Wall - 510kg/m³ - 1.95mm

Table K.9 - Span Table - Wall - 550kg/m3 - 0.40mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)	56.7	%			
Panel Width	600	mm	N3 Region Max		Shear (C)	43.3	%			
Core Density	550	kg/m^3	N5 Region Max		Bending (S)	0.0	%			
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	5.7	3.3	2.1	1.4	1.0	0.7	D	D
100mm	17.4	13.1	10.5	6.3	3.9	2.6	1.9	1.4	1.0	0.8
125mm	21.8	16.3	13.1	10.4	6.6	4.4	3.1	2.2	1.7	1.3
150mm	26.1	19.6	15.7	13.1	11.2	8.2	5.7	4.2	3.1	2.4
175mm	30.5	22.9	18.3	15.2	13.1	11.4	8.5	6.2	4.6	3.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	8.6	6.5	5.0

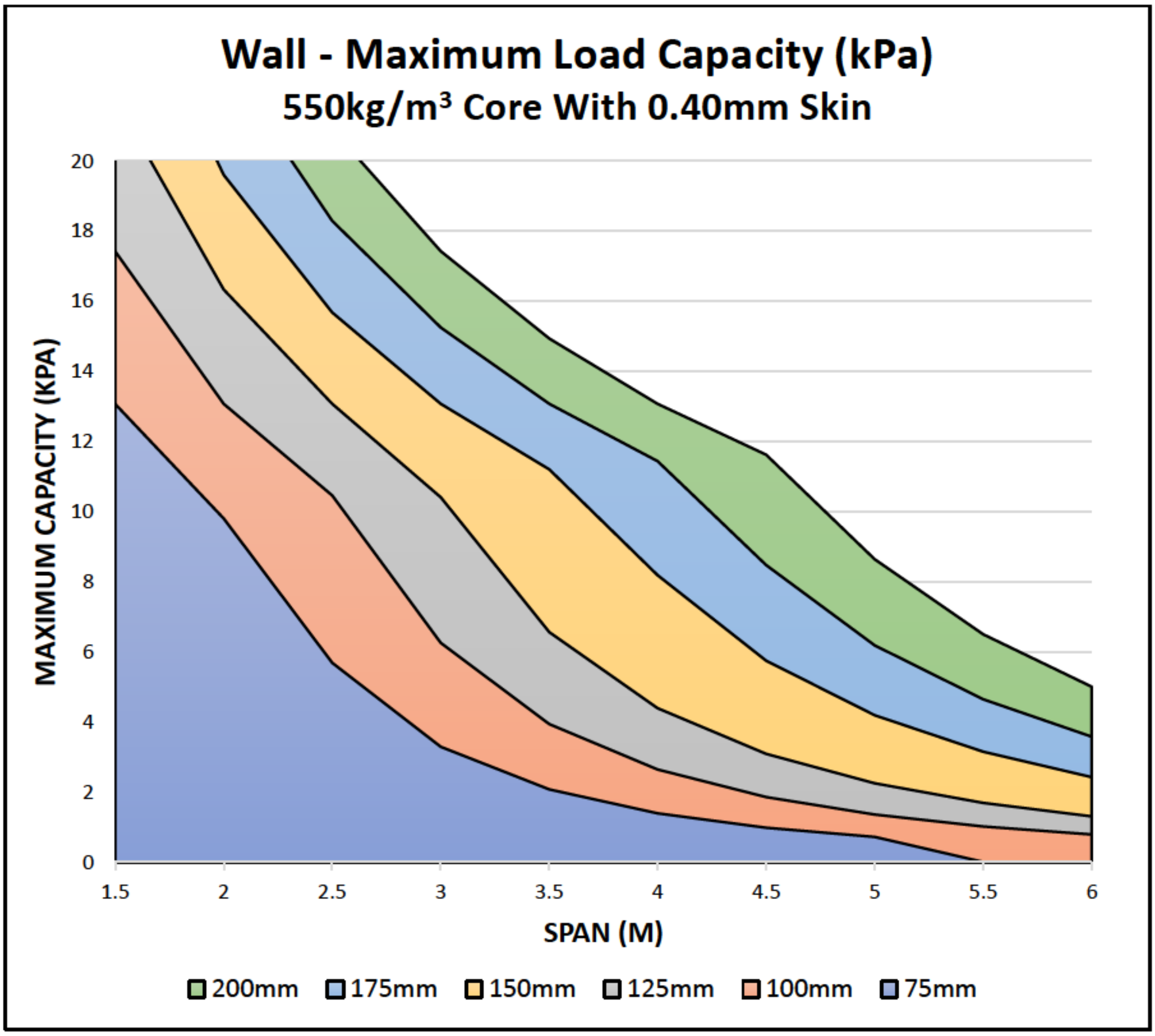


Figure K.9 - Load Capacity Graph - Wall - 550kg/m3 - 0.40mm

Table K.10 - Span Table - Wall - 550kg/m3 - 0.55mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55 mm	N1 Region Max	Limiting	Deflection (D)	53.3 %					
Panel Width	600 mm	N3 Region Max		Shear (C)	46.7 %					
Core Density	550 kg/m^3	N5 Region Max		Bending (S)	0.0 %					
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.4	4.3	2.7	1.8	1.3	0.9	0.7	D
100mm	17.4	13.1	10.5	8.0	5.0	3.4	2.4	1.7	1.3	1.0
125mm	21.8	16.3	13.1	10.9	8.3	5.5	3.9	2.8	2.1	1.6
150mm	26.1	19.6	15.7	13.1	11.2	9.8	6.9	5.0	3.8	2.9
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.0	7.3	5.5	4.2
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.1	7.6	5.9

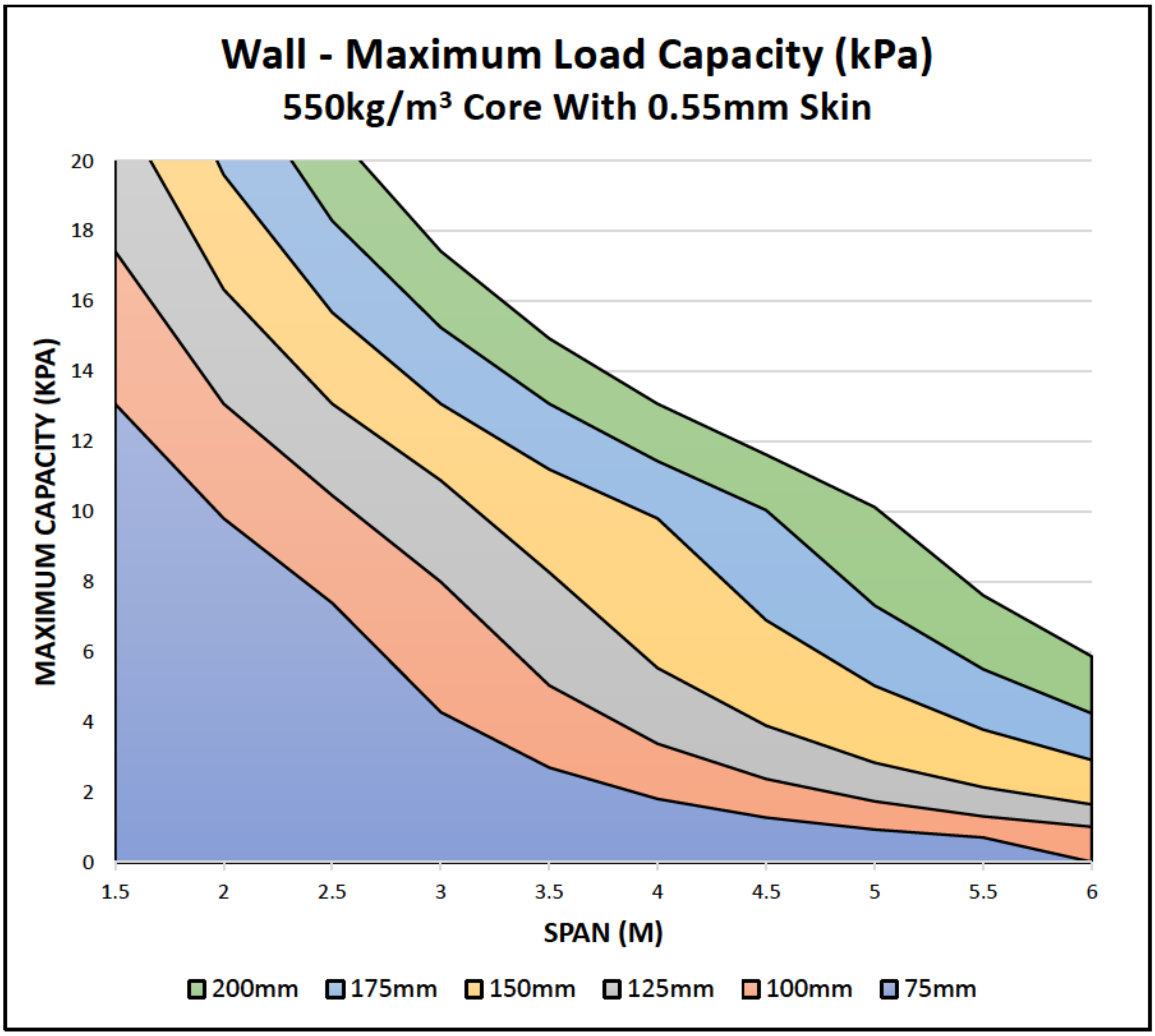


Figure K.10 - Load Capacity Graph - Wall - 550kg/m3 - 0.55mm

Table K.11 - Span Table - Wall - 550kg/m3 - 0.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max	Limiting	Deflection (D)		35.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		65.0	%		
Core Density	550	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	4.4	2.9	2.1	1.5	1.1	0.9
100mm	17.4	13.1	10.5	8.7	7.5	5.3	3.8	2.7	2.1	1.6
125mm	21.8	16.3	13.1	10.9	9.3	8.2	6.0	4.4	3.3	2.5
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.3	5.5	4.2
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	7.8	6.0
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.2

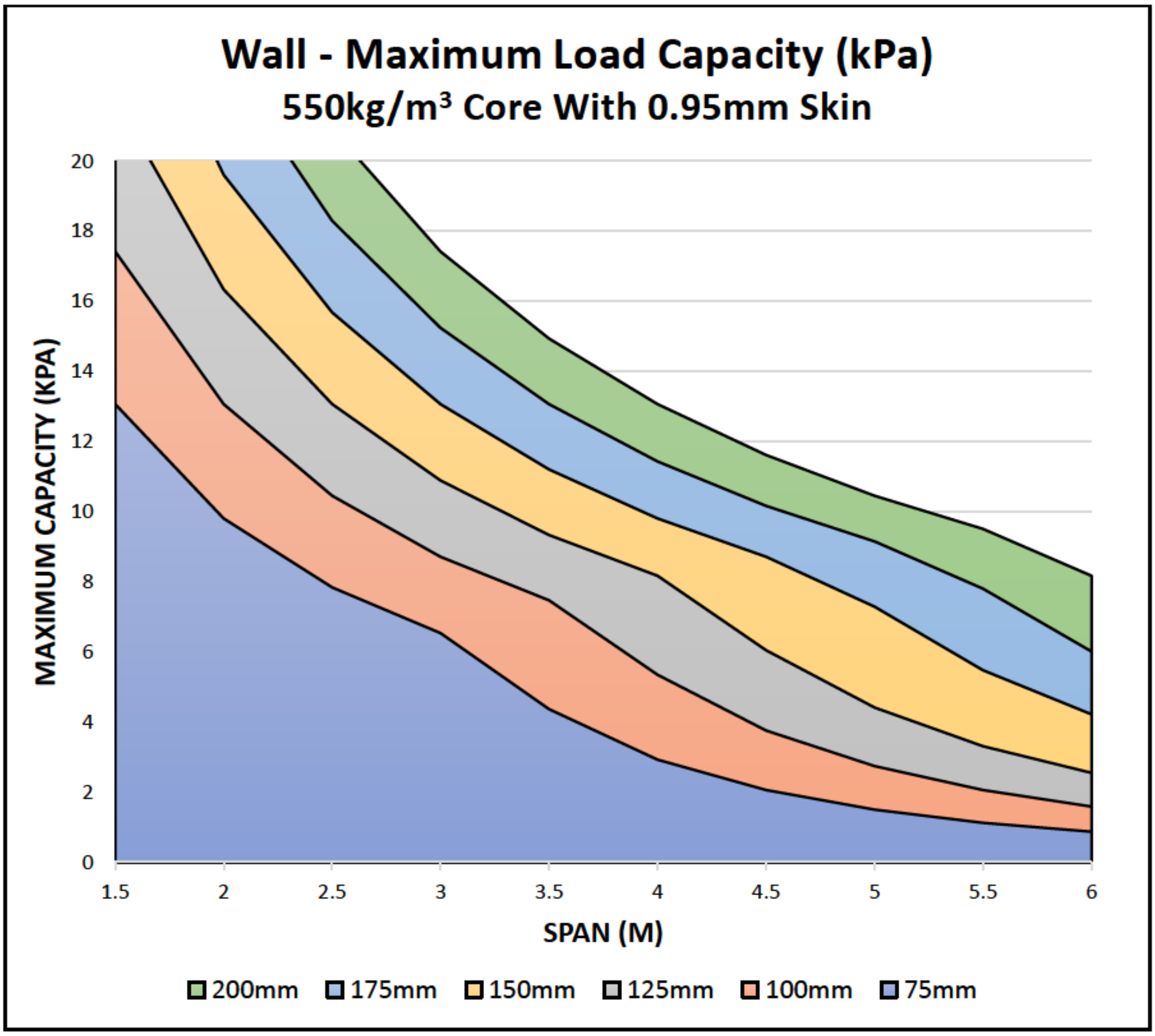


Figure K.11 - Load Capacity Graph - Wall - 550kg/m3 - 0.95mm

Table K.12 - Span Table - Wall - 550kg/m3 - 1.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)		11.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		88.3	%		
Core Density	550	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	5.6	4.9	4.1	3.0	2.2	1.7
100mm	17.4	13.1	10.5	8.7	7.5	6.5	5.8	5.2	4.0	3.1
125mm	21.8	16.3	13.1	10.9	9.3	8.2	7.3	6.5	5.9	4.9
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.8	7.1	6.5
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	8.3	7.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.7

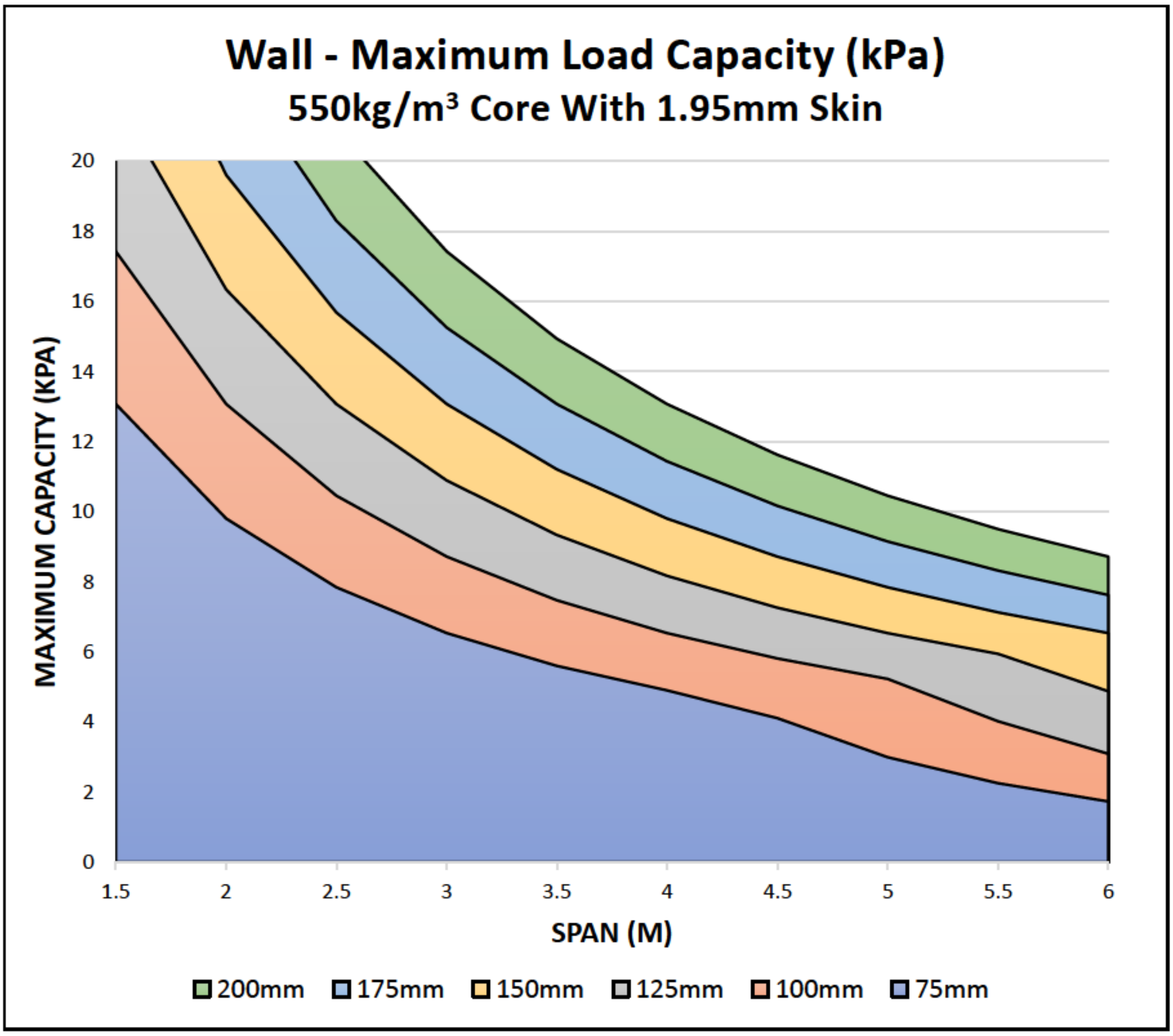


Figure K.12 - Load Capacity Graph - Wall - 550kg/m3 - 1.95mm

Table K.13 - Span Table - Wall - 580kg/m3 - 0.40mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)		56.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		43.3	%		
Core Density	580	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	5.8	3.3	2.1	1.4	1.0	0.7	D	D
100mm	17.4	13.1	10.5	6.4	4.0	2.7	1.9	1.4	1.0	0.8
125mm	21.8	16.3	13.1	10.6	6.7	4.5	3.1	2.3	1.7	1.3
150mm	26.1	19.6	15.7	13.1	11.2	8.3	5.9	4.3	3.2	2.5
175mm	30.5	22.9	18.3	15.2	13.1	11.4	8.7	6.3	4.7	3.7
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	8.8	6.6	5.1

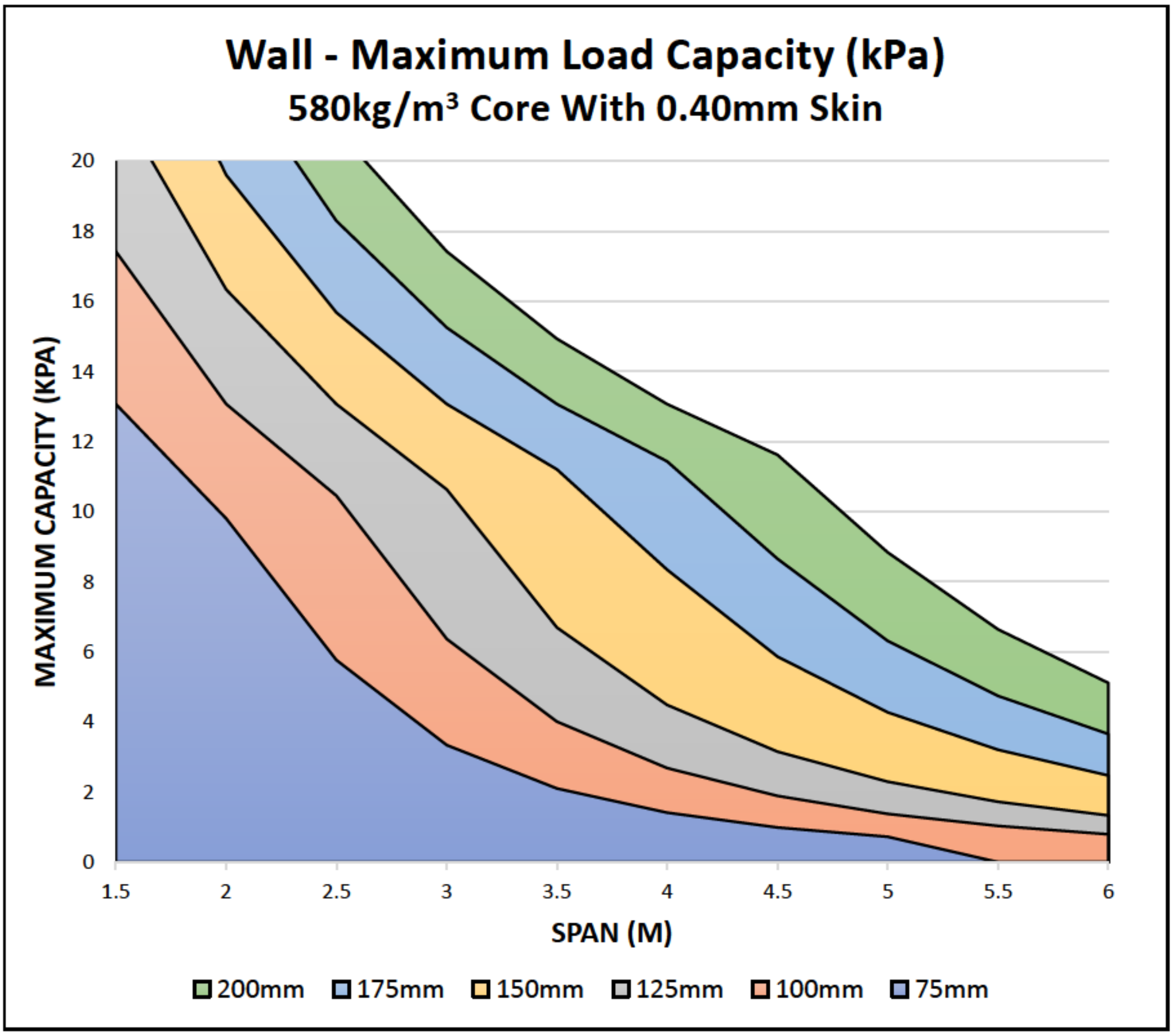


Figure K.13 - Load Capacity Graph - Wall - 580kg/m3 – 0.40mm

Table K.14 - Span Table - Wall - 580kg/m3 - 0.55mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)		51.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		48.3	%		
Core Density	580	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.5	4.3	2.7	1.8	1.3	0.9	0.7	D
100mm	17.4	13.1	10.5	8.1	5.1	3.4	2.4	1.8	1.3	1.0
125mm	21.8	16.3	13.1	10.9	8.4	5.6	4.0	2.9	2.2	1.7
150mm	26.1	19.6	15.7	13.1	11.2	9.8	7.0	5.1	3.8	3.0
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	7.4	5.6	4.3
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.3	7.8	6.0

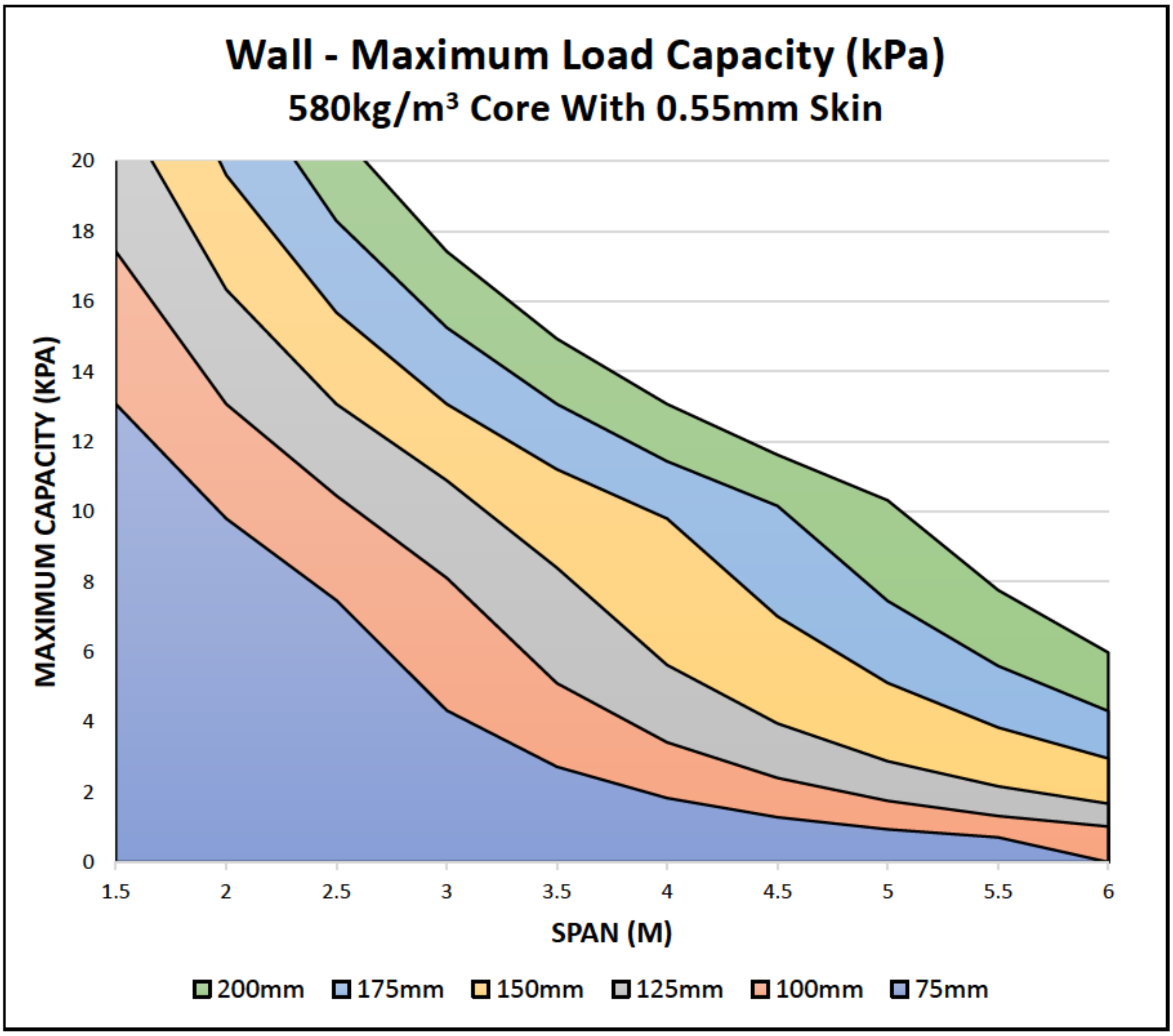


Figure K.14 - Load Capacity Graph - Wall - 580kg/m3 - 0.55mm

Table K.15 - Span Table - Wall - 580kg/m3 - 0.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max	Limiting	Deflection (D)		35.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		65.0	%		
Core Density	580	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	4.4	2.9	2.1	1.5	1.1	0.9
100mm	17.4	13.1	10.5	8.7	7.5	5.4	3.8	2.8	2.1	1.6
125mm	21.8	16.3	13.1	10.9	9.3	8.2	6.1	4.5	3.3	2.6
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.4	5.5	4.3
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	7.9	6.1
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.3

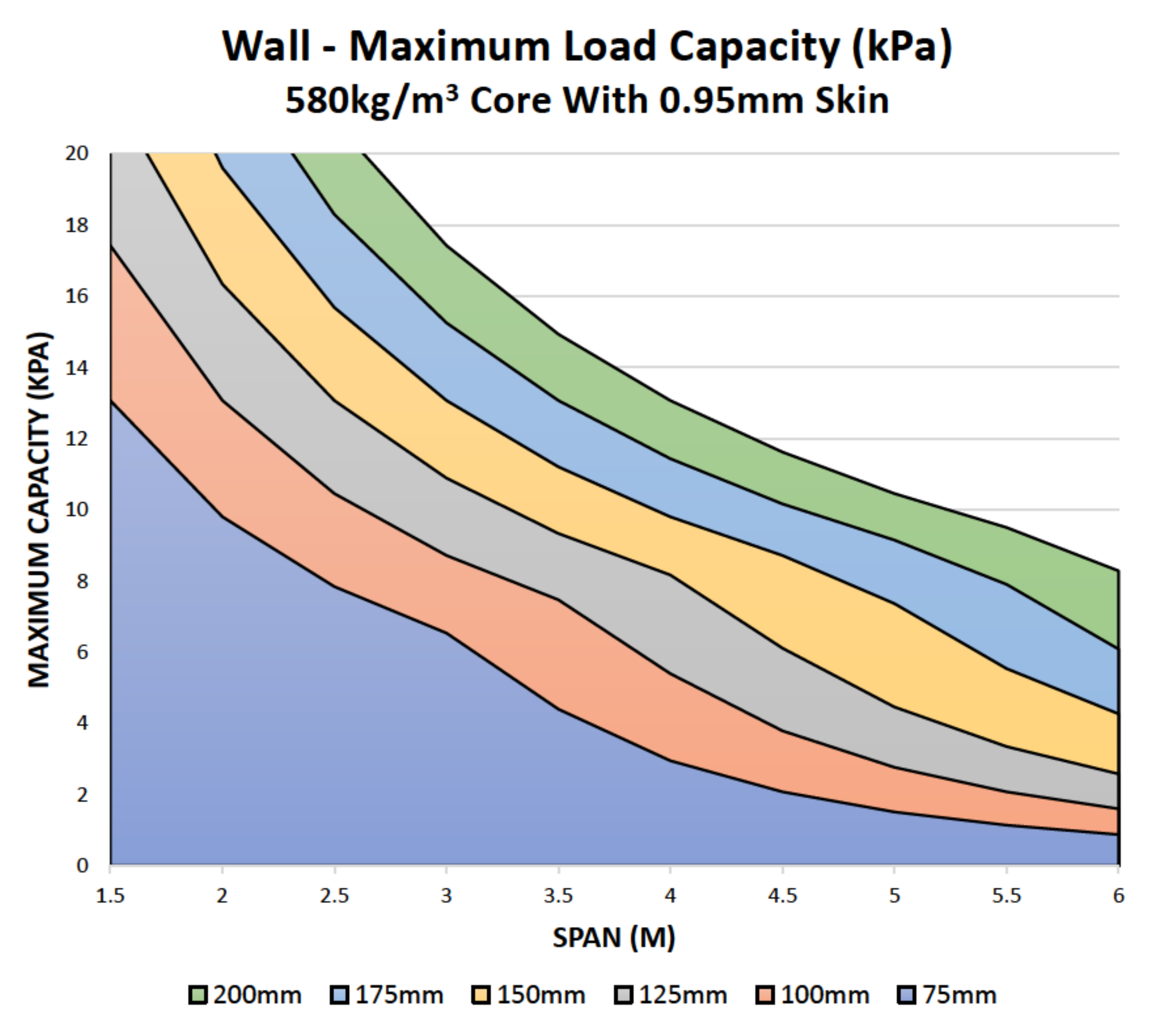


Figure K.15 - Load Capacity Graph - Wall - 580kg/m3 - 0.95mm

Table K.16 - Span Table - Wall - 580kg/m3 - 1.95mm

Wall Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)		11.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		88.3	%		
Core Density	580	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	**Span Length (m)**									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	5.6	4.9	4.1	3.0	2.3	1.7
100mm	17.4	13.1	10.5	8.7	7.5	6.5	5.8	5.2	4.0	3.1
125mm	21.8	16.3	13.1	10.9	9.3	8.2	7.3	6.5	5.9	4.9
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.8	7.1	6.5
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	8.3	7.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.7

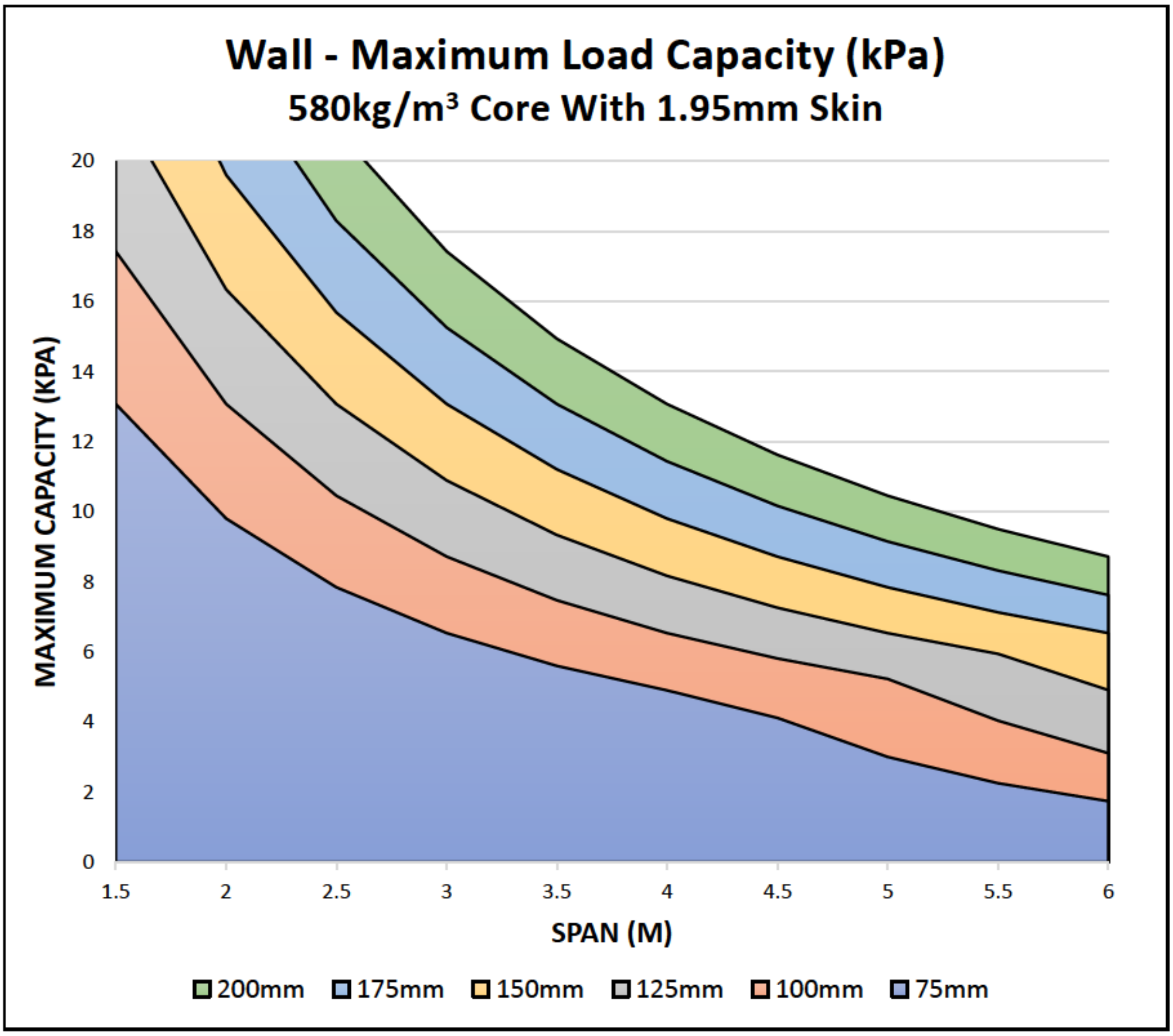


Figure K.16 - Load Capacity Graph - Wall - 580kg/m3 - 1.95mm

L Appendix L

Table L.1 - Span Table - Roof - 400kg/m³ - 0.40mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)		50.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		50.0	%		
Core Density	400	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.5	2.6	1.6	1.1	D	D	D	D
100mm	10.5	7.8	6.3	4.8	3.0	2.0	1.4	D	D	D
125mm	13.1	9.8	7.8	6.5	5.0	3.3	2.4	1.7	1.3	D
150mm	15.7	11.8	9.4	7.8	6.7	5.9	4.4	3.2	2.4	1.9
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	4.7	3.5	2.7
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	4.9	3.8

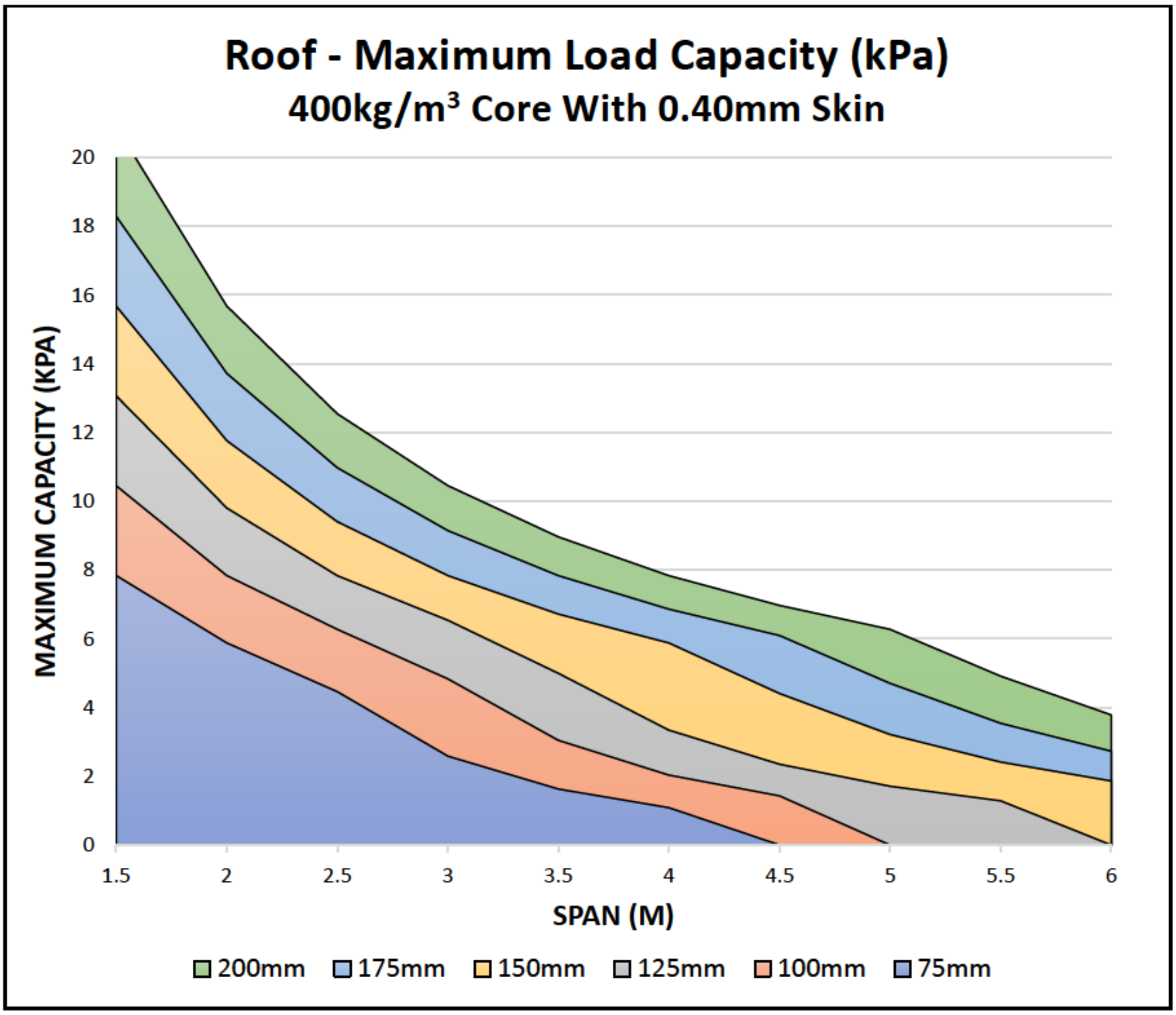


Figure L.1 - Load Capacity Graph - Roof - 400kg/m3 - 0.40mm

Table L.2 - Span Table - Roof - 400kg/m³ - 0.55mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)		40.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		60.0	%		
Core Density	400	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.7	3.4	2.1	1.4	D	D	D	D
100mm	10.5	7.8	6.3	5.2	4.0	2.6	1.9	1.4	D	D
125mm	13.1	9.8	7.8	6.5	5.6	4.3	3.0	2.2	1.7	1.3
150mm	15.7	11.8	9.4	7.8	6.7	5.9	5.2	3.9	2.9	2.3
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	4.3	3.3
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	4.5

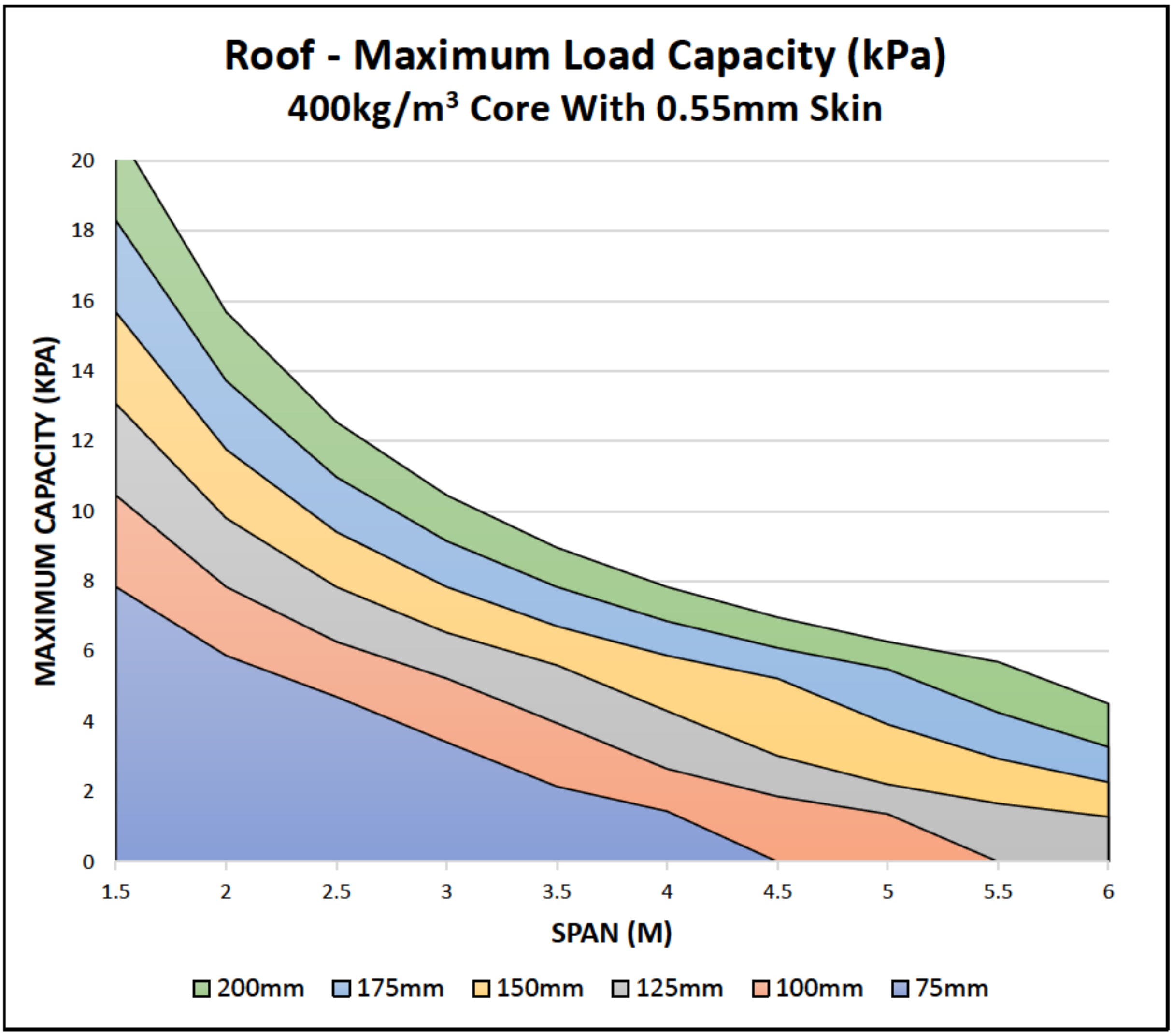


Figure L.2 - Load Capacity Graph - Roof - 400kg/m3 - 0.55mm

Table L.3 - Span Table - Roof - 400kg/m3 - 0.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max	Limiting	Deflection (D)		21.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		78.3	%		
Core Density	400	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.7	3.9	3.4	2.4	1.7	1.2	D	D
100mm	10.5	7.8	6.3	5.2	4.5	3.9	3.0	2.2	1.7	1.3
125mm	13.1	9.8	7.8	6.5	5.6	4.9	4.4	3.5	2.6	2.0
150mm	15.7	11.8	9.4	7.8	6.7	5.9	5.2	4.7	4.3	3.4
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	4.6
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	5.2

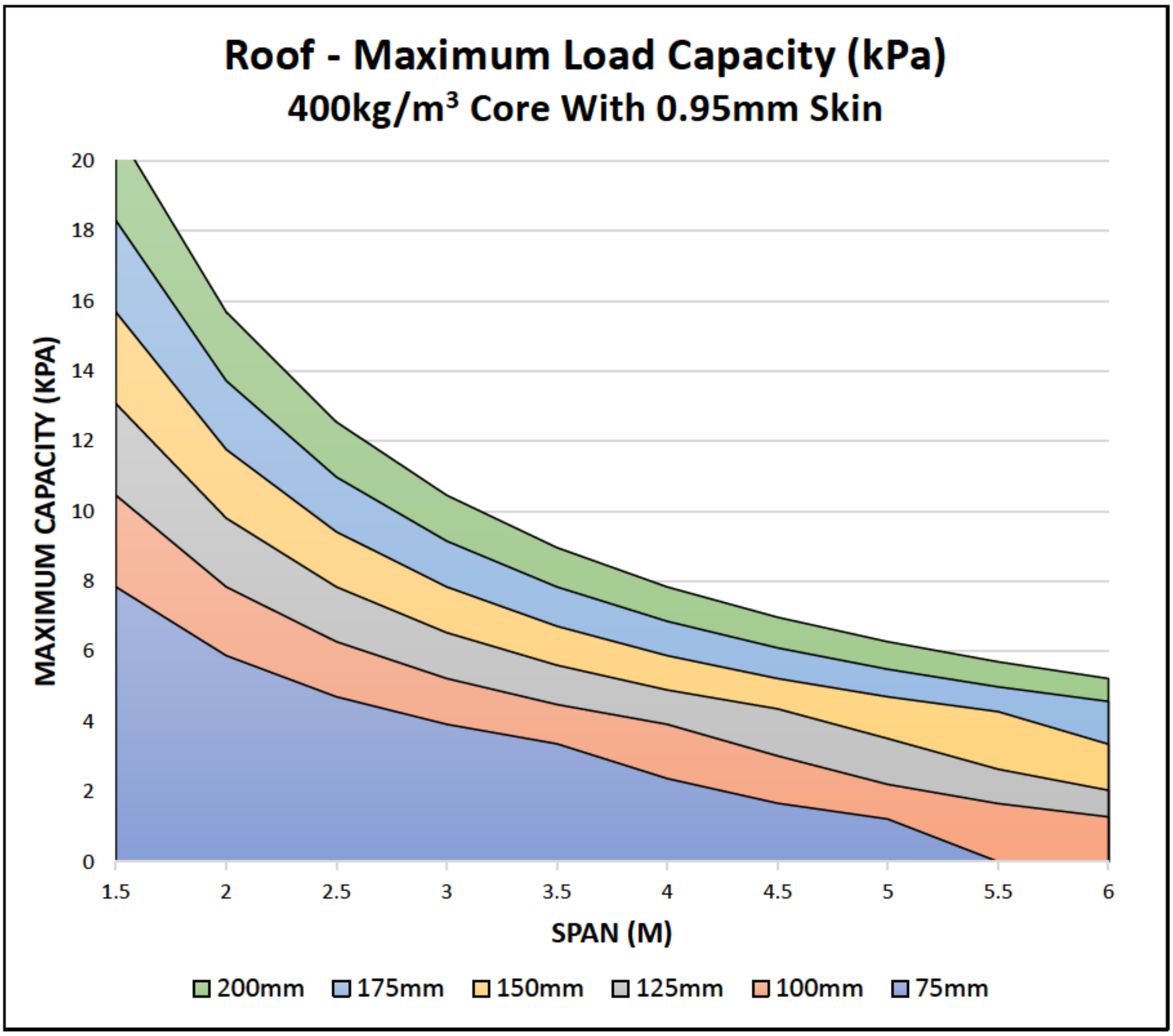


Figure L.3 - Load Capacity Graph - Roof - 400kg/m3 - 0.95mm

Table L.4 - Span Table - Roof - 400kg/m³ - 1.95mm

Roof Application Maximum Load Capacity (kPa) Table
Single Span – Limited by Serviceability & Ultimate Limit State

Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)	5.0	%
Panel Width	600	mm	N3 Region Max		Shear (C)	95.0	%
Core Density	400	kg/m³	N5 Region Max		Bending (S)	0.0	%

Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	4.7	3.9	3.4	2.9	2.6	2.4	1.8	1.4
100mm	10.5	7.8	6.3	5.2	4.5	3.9	3.5	3.1	2.9	2.5
125mm	13.1	9.8	7.8	6.5	5.6	4.9	4.4	3.9	3.6	3.3
150mm	15.7	11.8	9.4	7.8	6.7	5.9	5.2	4.7	4.3	3.9
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	4.6
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	5.2

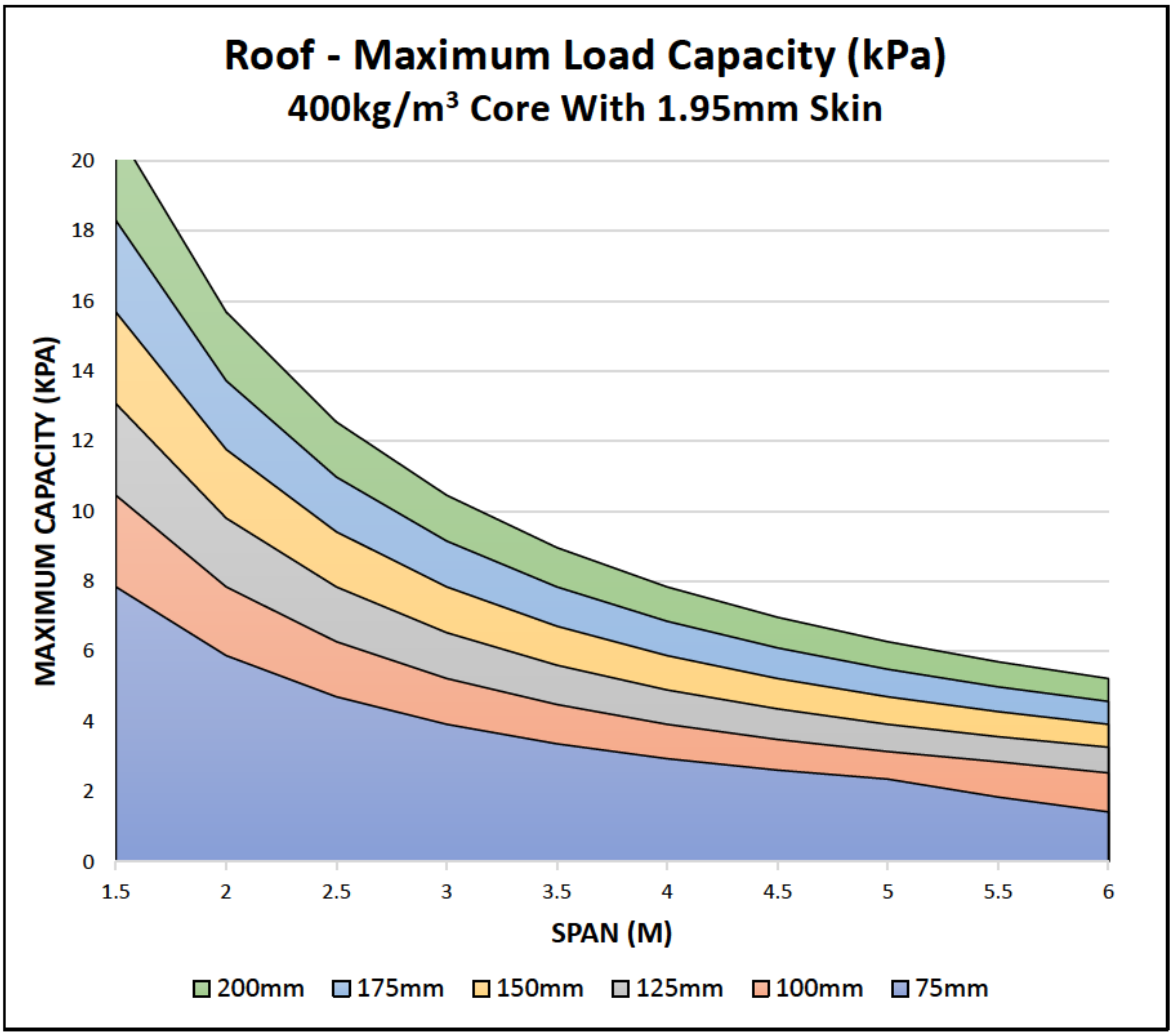


Figure L.4 - Load Capacity Graph - Roof - 400kg/m3 - 1.95mm

Table L.5 - Span Table - Roof - 510kg/m³ - 0.40mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)		55.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		45.0	%		
Core Density	510	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	4.6	2.7	1.7	1.1	D	D	D	D
100mm	12.2	9.1	7.3	5.1	3.2	2.1	1.5	1.1	D	D
125mm	15.2	11.4	9.1	7.6	5.3	3.6	2.5	1.8	1.4	D
150mm	18.3	13.7	11.0	9.1	7.8	6.6	4.7	3.4	2.6	2.0
175mm	21.3	16.0	12.8	10.7	9.1	8.0	6.9	5.0	3.8	2.9
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.0	5.2	4.0

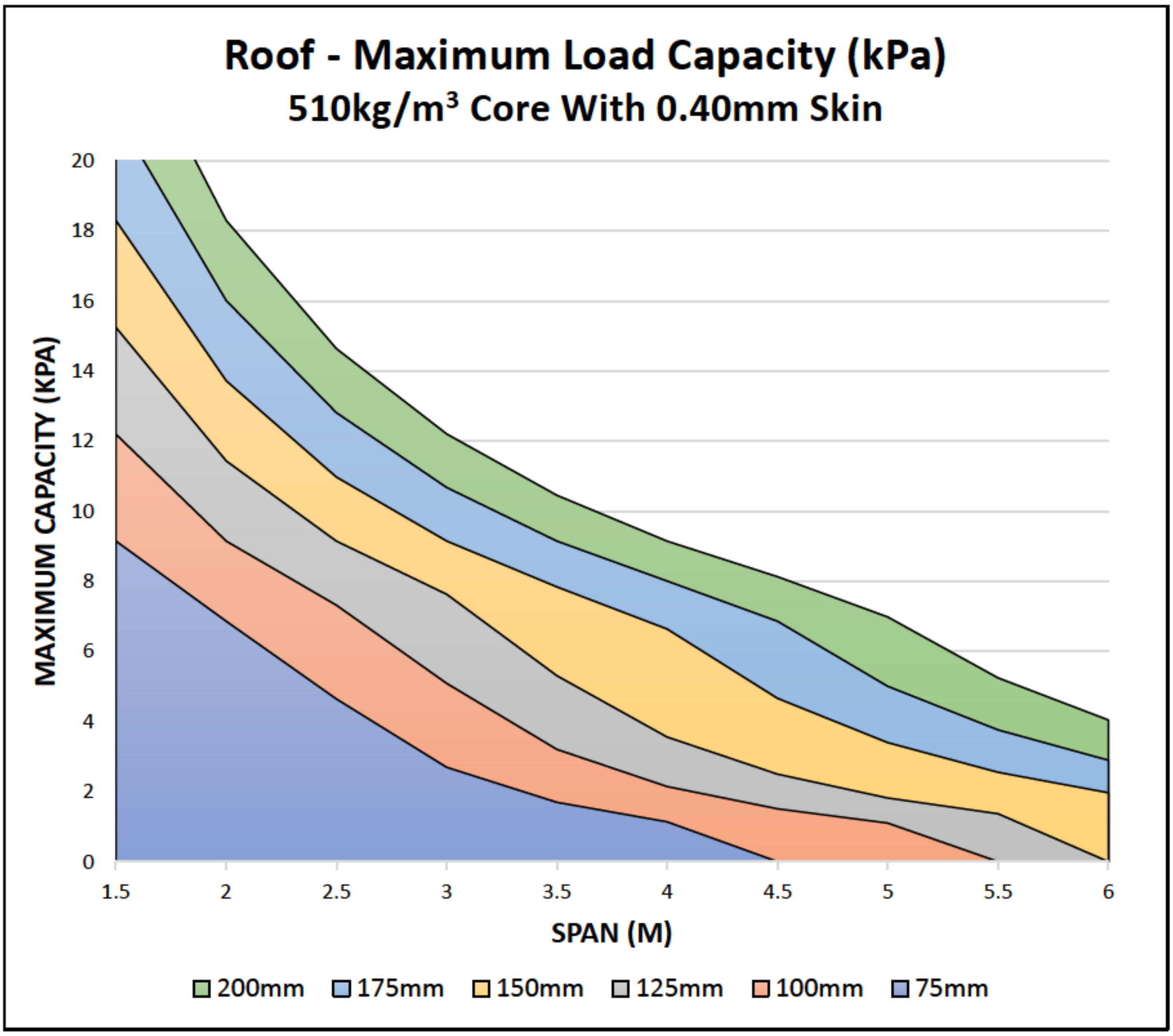


Figure L.5 - Load Capacity Graph - Roof - 510kg/m3 - 0.40mm

Table L.6 - Span Table - Roof - 510kg/m³ - 0.55mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)		45.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		55.0	%		
Core Density	510	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	3.5	2.2	1.5	D	D	D	D
100mm	12.2	9.1	7.3	6.1	4.1	2.8	1.9	1.4	1.1	D
125mm	15.2	11.4	9.1	7.6	6.5	4.5	3.2	2.3	1.7	1.3
150mm	18.3	13.7	11.0	9.1	7.8	6.9	5.6	4.1	3.1	2.4
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.0	4.5	3.4
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.2	4.8

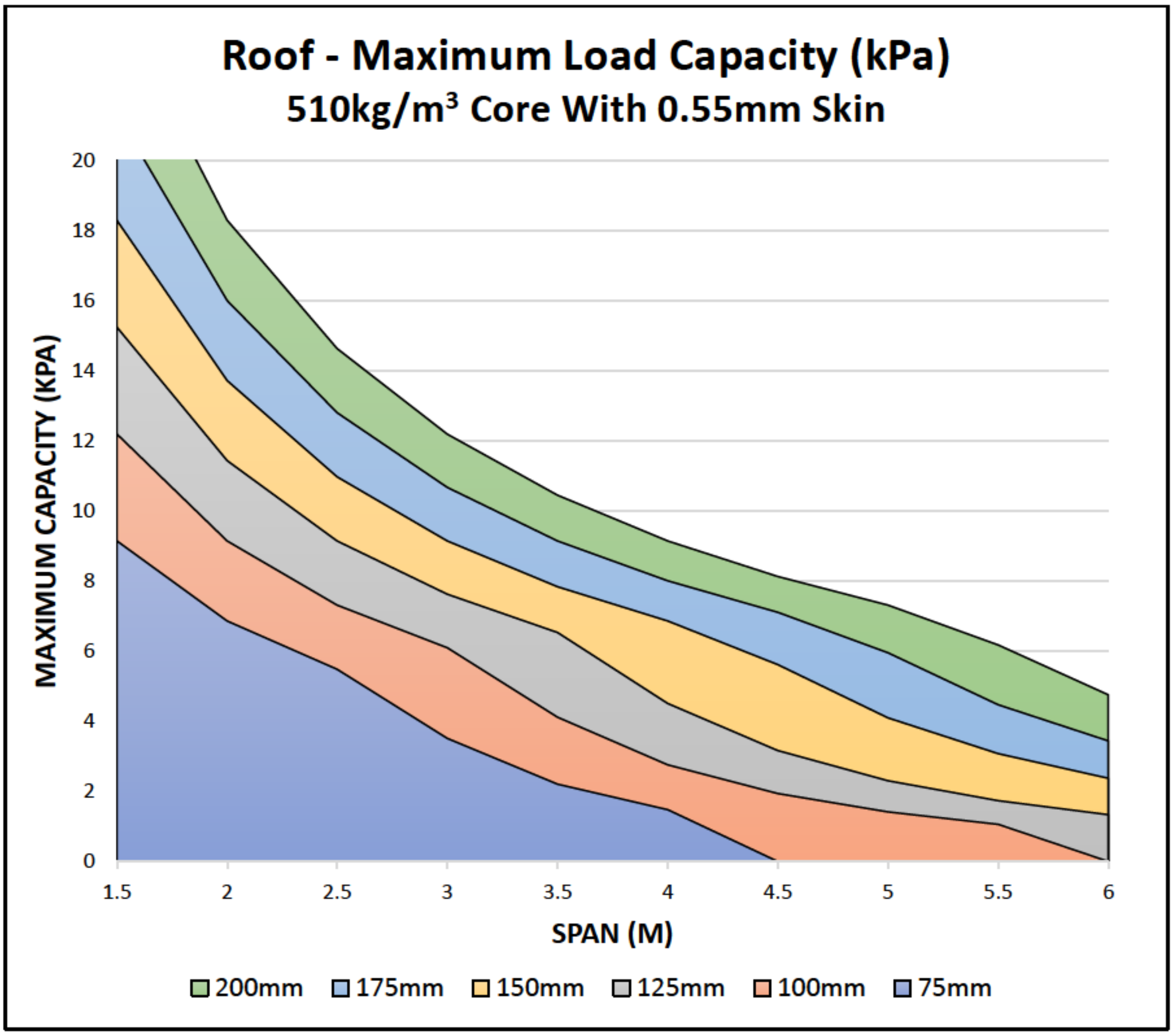


Figure L.6 - Load Capacity Graph - Roof - 510kg/m3 - 0.55mm

Table L.7 - Span Table - Roof - 510kg/m³ - 0.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max		Limiting	Deflection (D)		30.0	%	
Panel Width	600	mm	N3 Region Max			Shear (C)		70.0	%	
Core Density	510	kg/m³	N5 Region Max			Bending (S)		0.0	%	
Core Thickness	**Span Length (m)**									
	1.5	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**	**6**
75mm	9.1	6.9	5.5	4.6	3.6	2.4	1.7	1.2	D	D
100mm	12.2	9.1	7.3	6.1	5.2	4.4	3.1	2.3	1.7	1.3
125mm	15.2	11.4	9.1	7.6	6.5	5.7	5.0	3.6	2.7	2.1
150mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	4.5	3.5
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.4	5.8	4.9
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.7	6.1

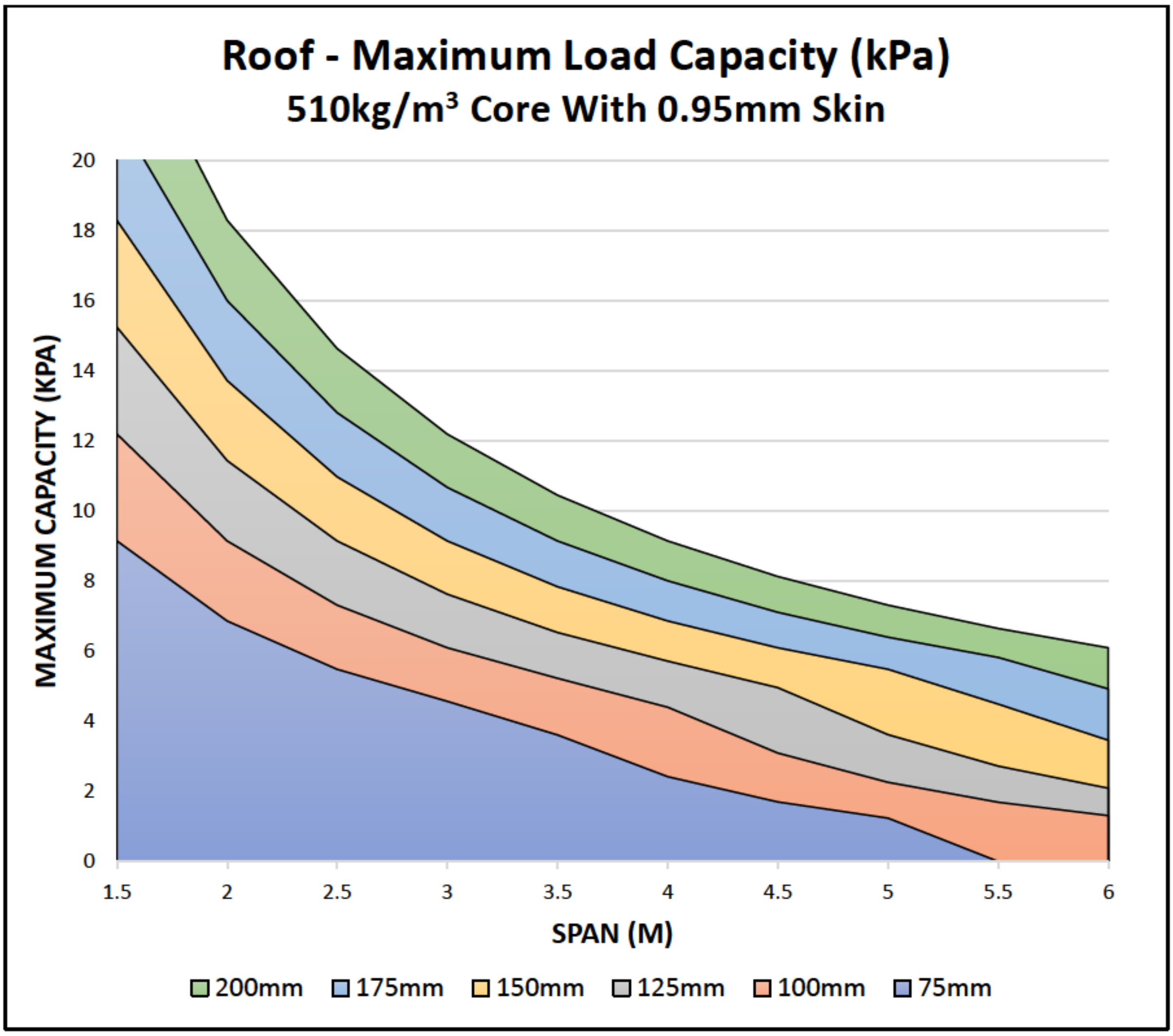


Figure L.7 - Load Capacity Graph - Roof - 510kg/m3 - 0.95mm

Table L.8 - Span Table - Roof - 510kg/m³ - 1.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)		8.3	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		91.7	%		
Core Density	510	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	4.6	3.9	3.4	3.0	2.5	1.9	1.4
100mm	12.2	9.1	7.3	6.1	5.2	4.6	4.1	3.7	3.3	2.6
125mm	15.2	11.4	9.1	7.6	6.5	5.7	5.1	4.6	4.2	3.8
150mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	5.5	5.0	4.6
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.4	5.8	5.3
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.7	6.1

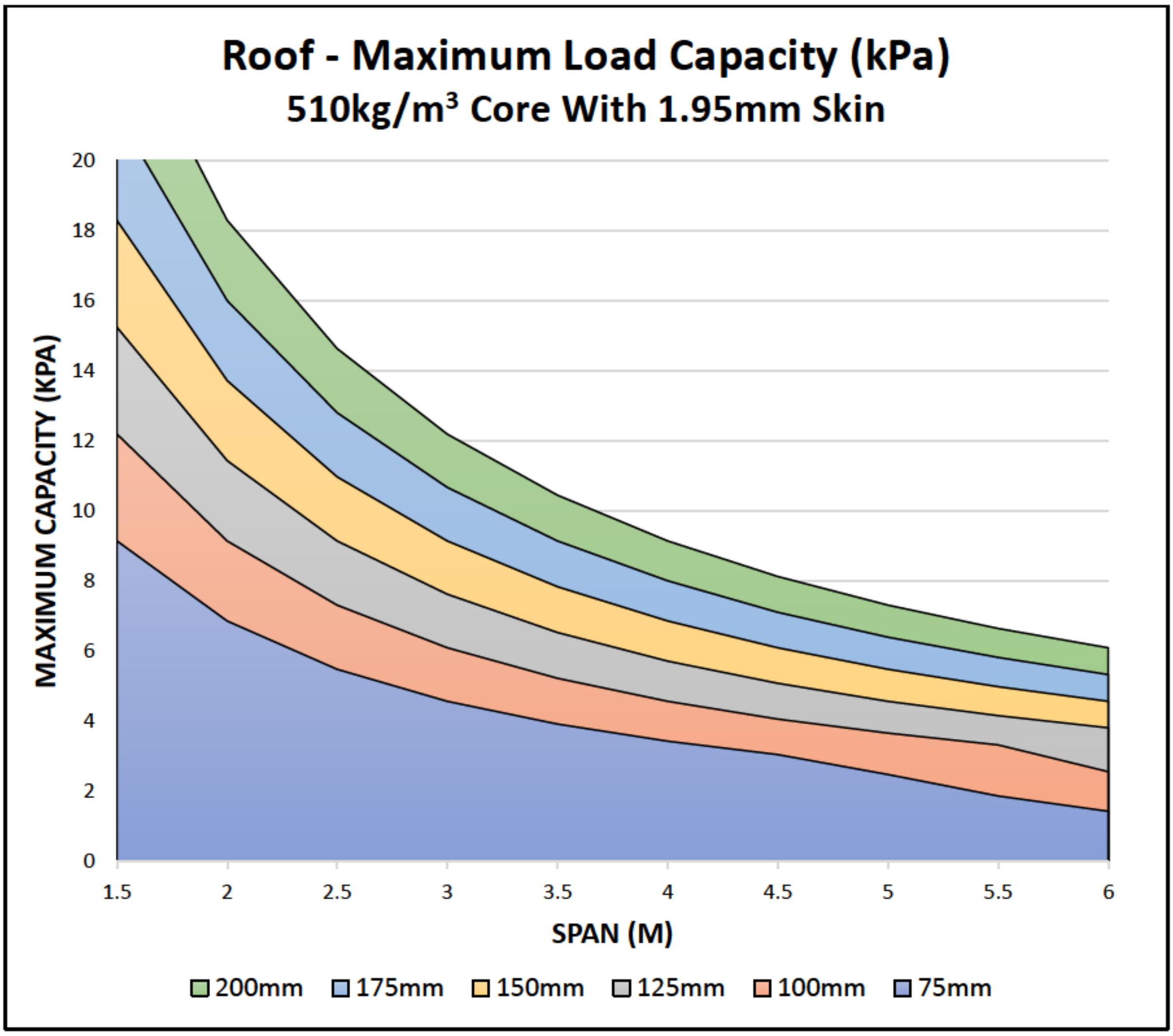


Figure L.8 - Load Capacity Graph - Roof - 510kg/m3 - 1.95mm

Table L.9 - Span Table - Roof - 550kg/m3 - 0.40mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)		65.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		35.0	%		
Core Density	550	kg/m^3	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.3	4.7	2.7	1.7	1.2	D	D	D	D
100mm	17.4	13.1	9.0	5.2	3.3	2.2	1.5	1.1	D	D
125mm	21.8	16.3	13.1	8.7	5.5	3.7	2.6	1.9	1.4	1.1
150mm	26.1	19.6	15.7	13.1	10.2	6.8	4.8	3.5	2.6	2.0
175mm	30.5	22.9	18.3	15.2	13.1	10.1	7.1	5.1	3.9	3.0
200mm	34.8	26.1	20.9	17.4	14.9	13.1	9.9	7.2	5.4	4.2

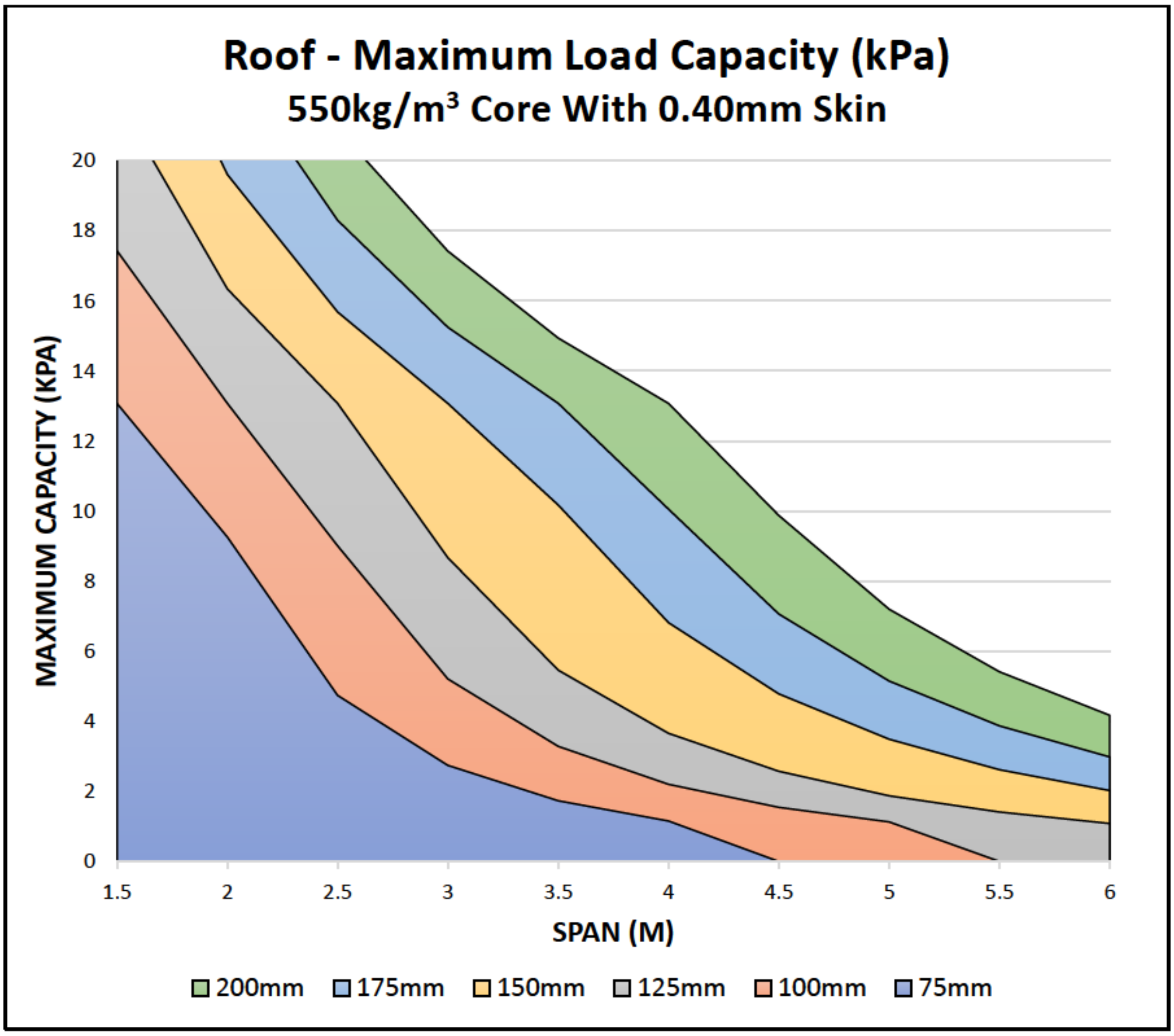


Figure L.9 - Load Capacity Graph - Roof - 550kg/m3 - 0.40mm

Table L.10 - Span Table - Roof - 550kg/m3 - 0.55mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)		56.7	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		43.3	%		
Core Density	550	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	6.2	3.6	2.2	1.5	D	D	D	D
100mm	17.4	13.1	10.5	6.7	4.2	2.8	2.0	1.4	1.1	D
125mm	21.8	16.3	13.1	10.9	6.9	4.6	3.2	2.4	1.8	1.4
150mm	26.1	19.6	15.7	13.1	11.2	8.2	5.7	4.2	3.1	2.4
175mm	30.5	22.9	18.3	15.2	13.1	11.4	8.4	6.1	4.6	3.5
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	8.4	6.3	4.9

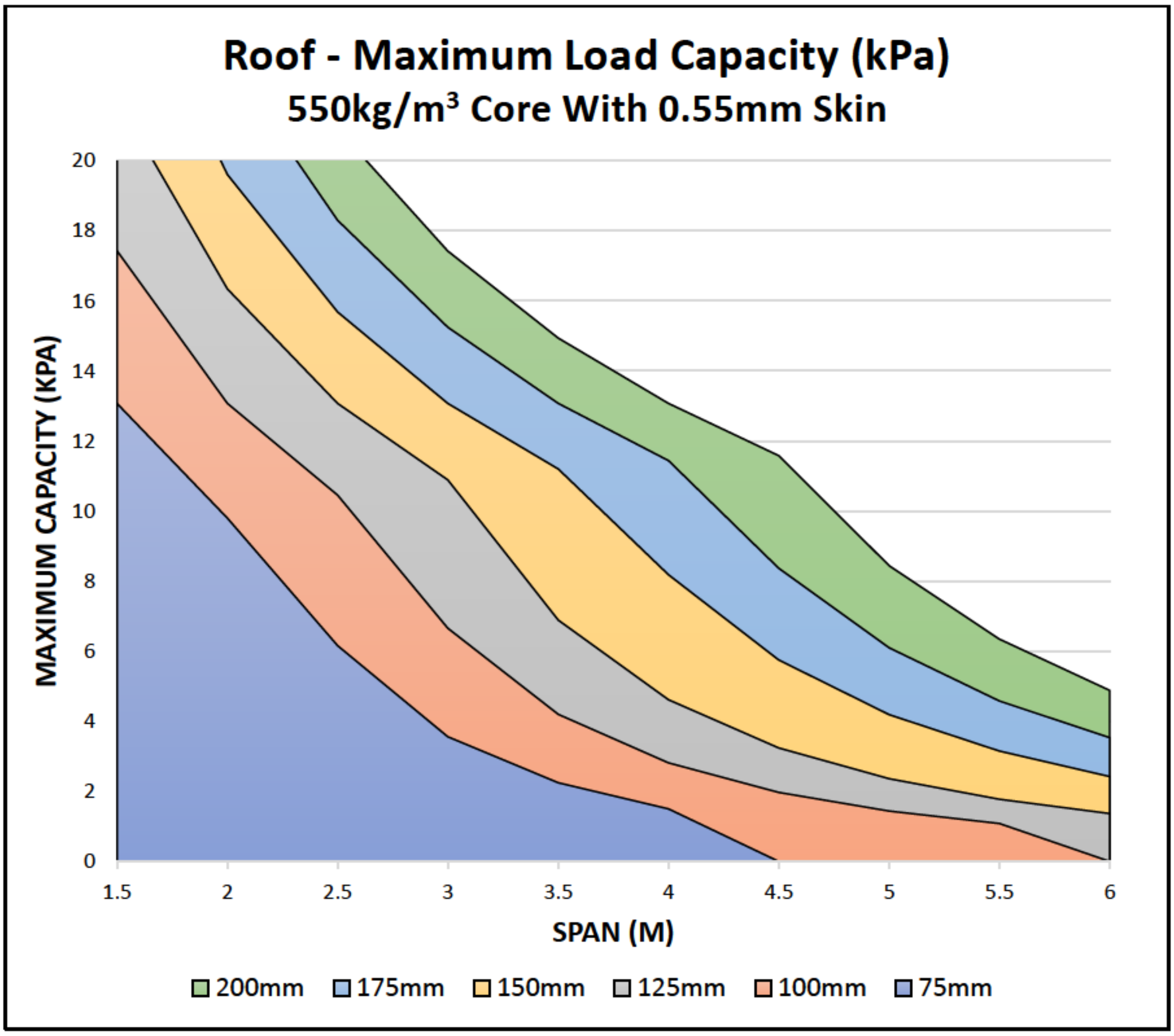


Figure L.10 - Load Capacity Graph - Roof - 550kg/m3 - 0.55mm

Table L.11 - Span Table - Roof - 550kg/m³ - 0.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max	Limiting	Deflection (D)	45.0	%			
Panel Width	600	mm	N3 Region Max		Shear (C)	55.0	%			
Core Density	550	kg/m³	N5 Region Max		Bending (S)	0.0	%			
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	5.8	3.6	2.4	1.7	1.2	D	D
100mm	17.4	13.1	10.5	8.7	6.7	4.5	3.1	2.3	1.7	1.3
125mm	21.8	16.3	13.1	10.9	9.3	7.2	5.0	3.7	2.8	2.1
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.3	6.1	4.6	3.5
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	8.6	6.5	5.0
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	8.8	6.8

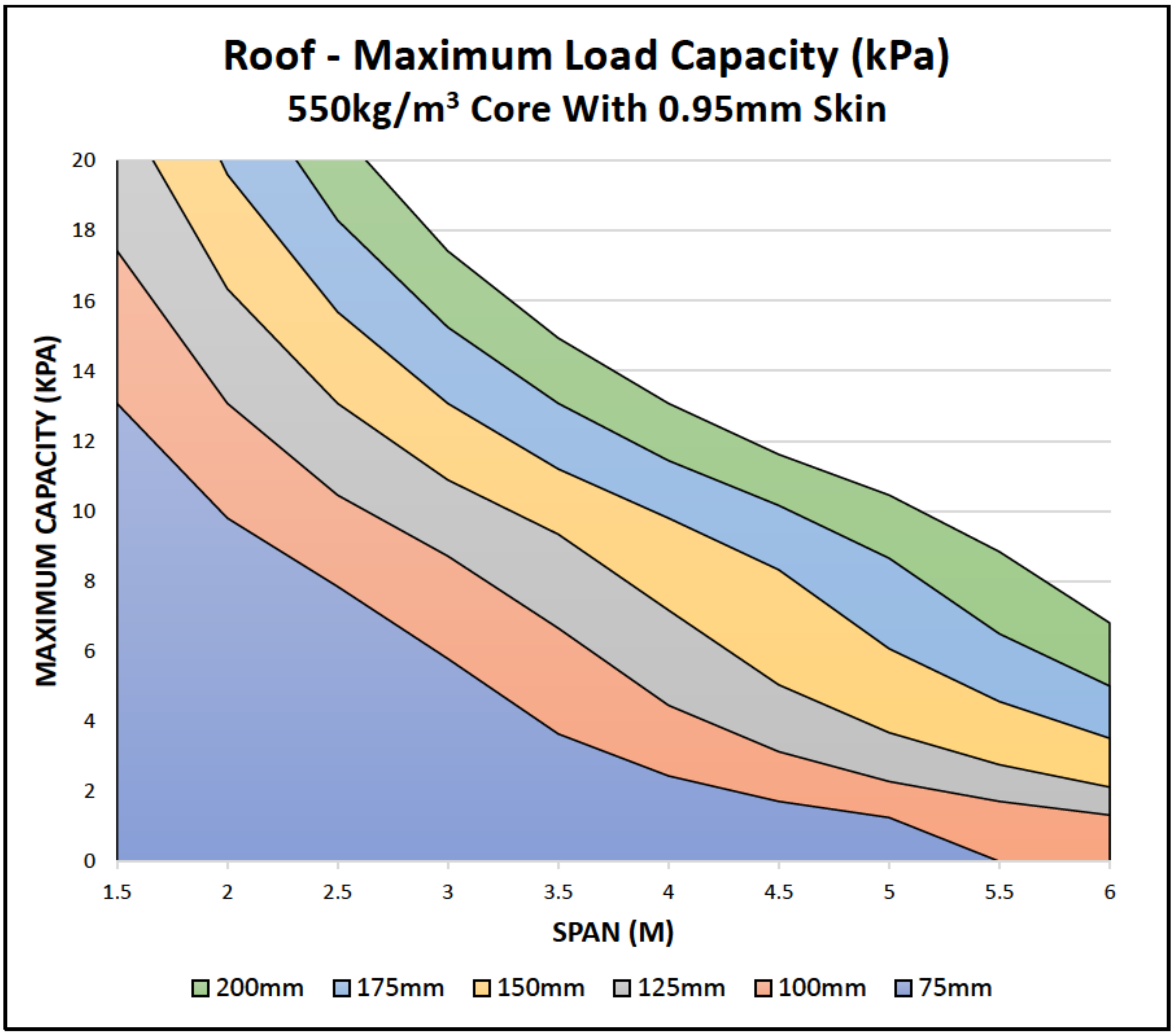


Figure L.11 - Load Capacity Graph - Roof - 550kg/m3 - 0.95mm

Table L.12 - Span Table - Roof - 550kg/m³ - 1.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)		18.3	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		81.7	%		
Core Density	550	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	5.6	4.9	3.4	2.5	1.9	1.4
100mm	17.4	13.1	10.5	8.7	7.5	6.5	5.8	4.5	3.3	2.6
125mm	21.8	16.3	13.1	10.9	9.3	8.2	7.3	6.5	5.3	4.1
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.8	7.1	6.3
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	8.3	7.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.7

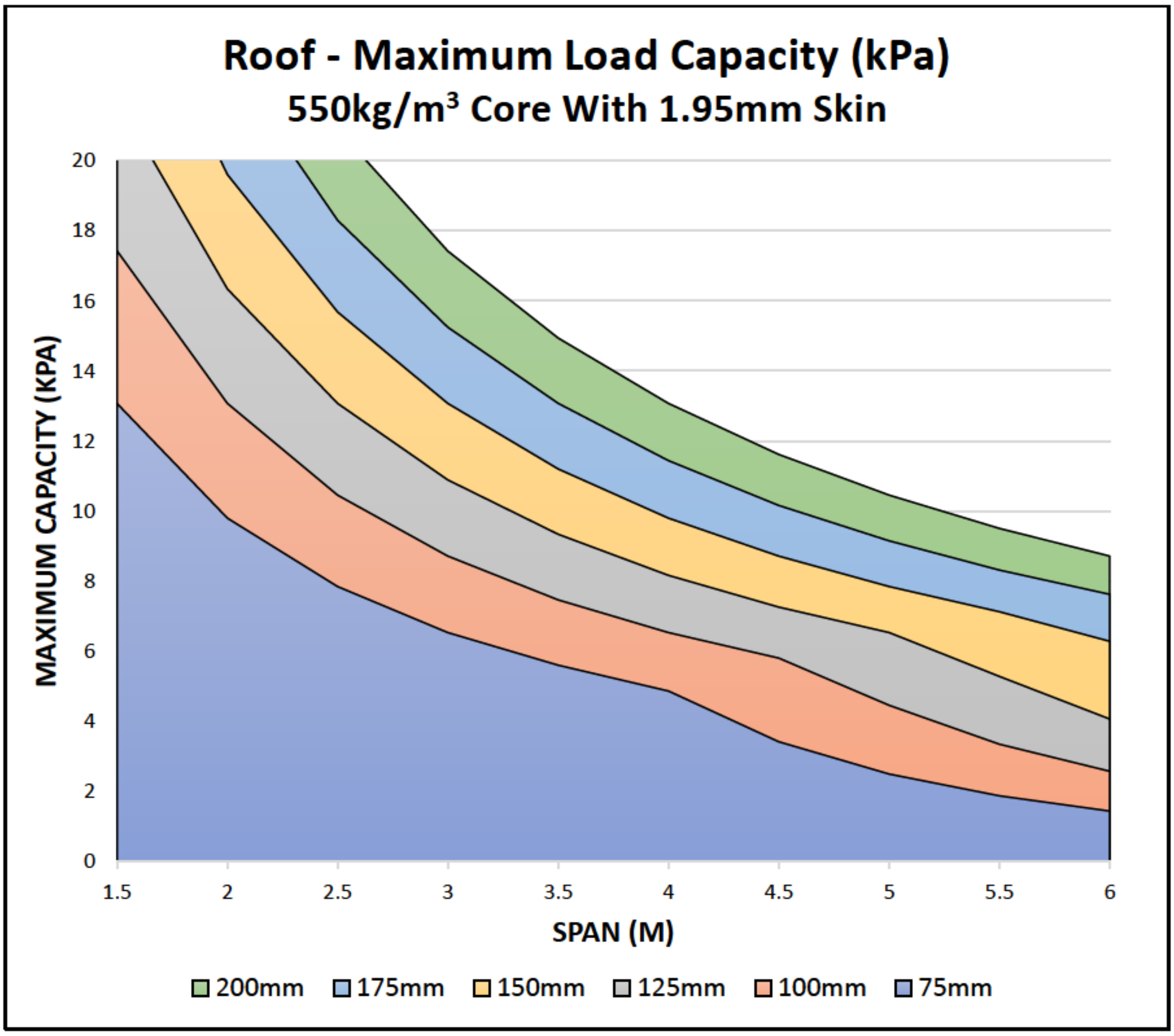


Figure L.12 - Load Capacity Graph - Roof - 550kg/m3 - 1.95mm

Table L.13 - Span Table - Roof - 580kg/m³ - 0.40mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm	N1 Region Max	Limiting	Deflection (D)	65.0	%			
Panel Width	600	mm	N3 Region Max		Shear (C)	35.0	%			
Core Density	580	kg/m³	N5 Region Max		Bending (S)	0.0	%			
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.4	4.8	2.8	1.8	1.2	D	D	D	D
100mm	17.4	13.1	9.2	5.3	3.3	2.2	1.6	1.1	D	D
125mm	21.8	16.3	13.1	8.9	5.6	3.7	2.6	1.9	1.4	1.1
150mm	26.1	19.6	15.7	13.1	10.4	7.0	4.9	3.6	2.7	2.1
175mm	30.5	22.9	18.3	15.2	13.1	10.3	7.2	5.3	3.9	3.0
200mm	34.8	26.1	20.9	17.4	14.9	13.1	10.1	7.4	5.5	4.3

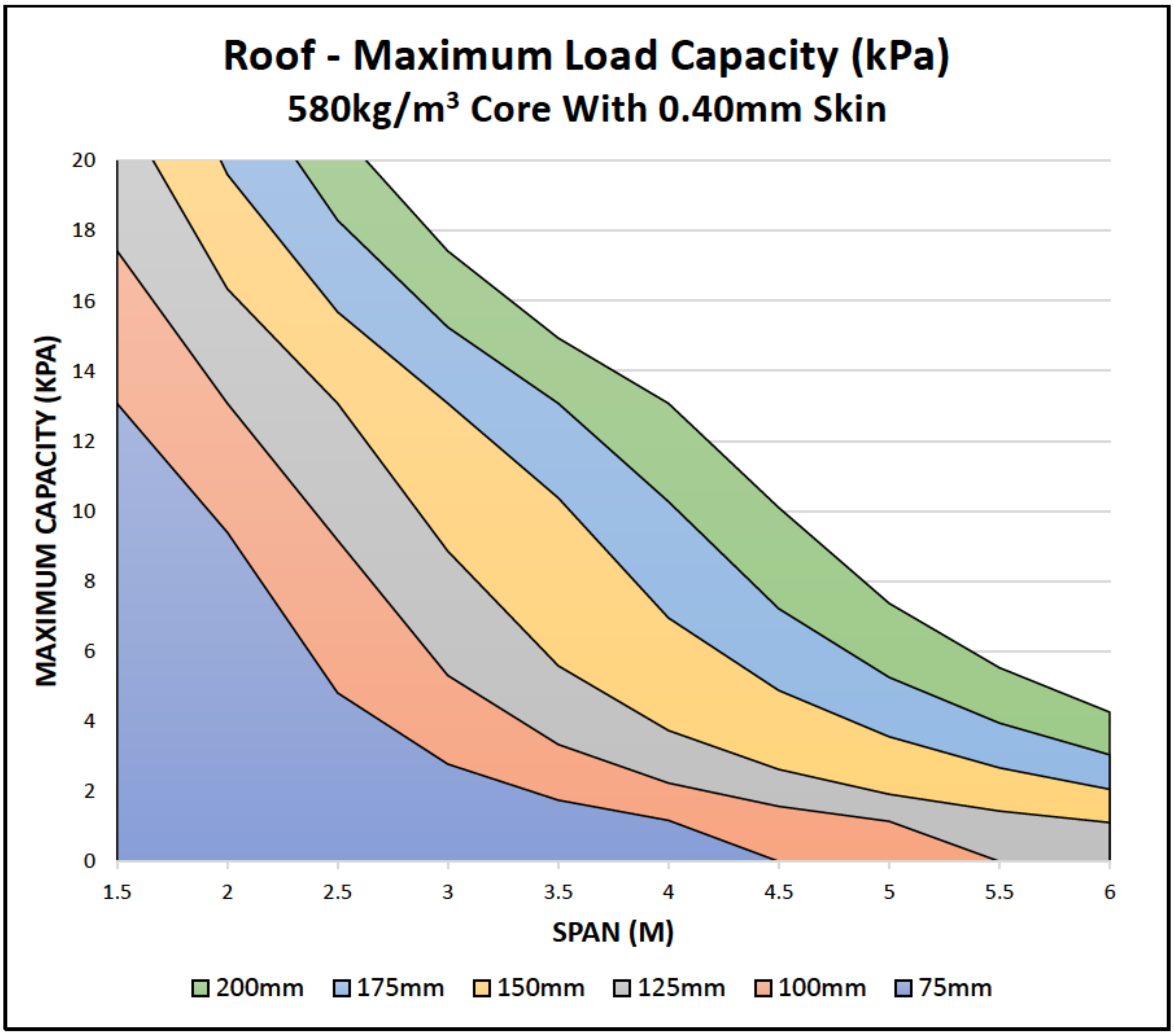


Figure L.13 - Load Capacity Graph - Roof - 580kg/m3 - 0.40mm

Table L.14 - Span Table - Roof - 580kg/m³ - 0.55mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm	N1 Region Max	Limiting	Deflection (D)			55.0	%	
Panel Width	600	mm	N3 Region Max		Shear (C)			45.0	%	
Core Density	580	kg/m³	N5 Region Max		Bending (S)			0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	6.2	3.6	2.3	1.5	1.1	D	D	D
100mm	17.4	13.1	10.5	6.8	4.3	2.8	2.0	1.5	1.1	D
125mm	21.8	16.3	13.1	10.9	7.0	4.7	3.3	2.4	1.8	1.4
150mm	26.1	19.6	15.7	13.1	11.2	8.3	5.8	4.3	3.2	2.5
175mm	30.5	22.9	18.3	15.2	13.1	11.4	8.5	6.2	4.7	3.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	8.6	6.5	5.0

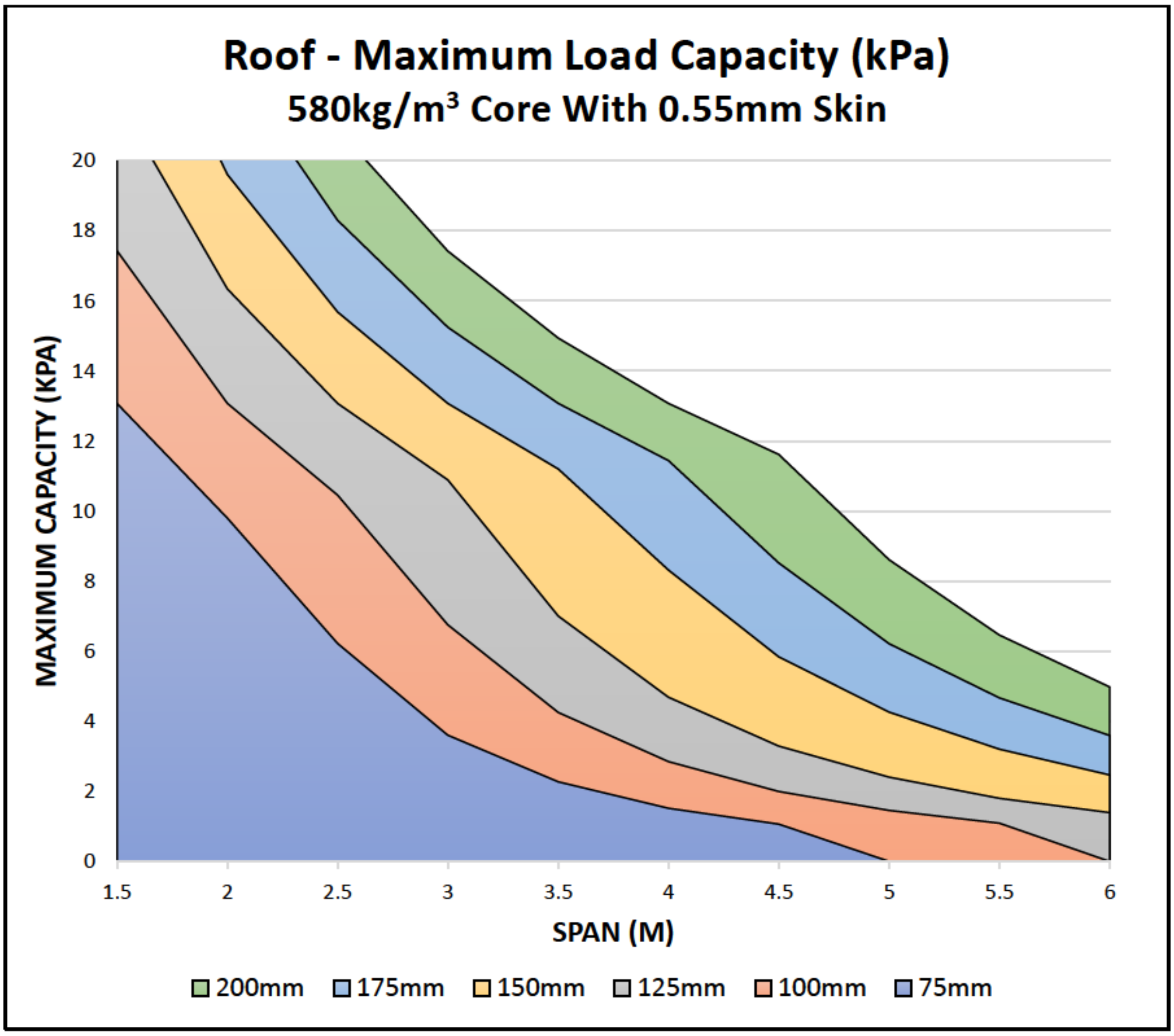


Figure L.14 - Load Capacity Graph - Roof - 580kg/m3 - 0.55mm

Table L.15 - Span Table - Roof - 580kg/m³ - 0.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm	N1 Region Max	Limiting	Deflection (D)		45.0	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		55.0	%		
Core Density	580	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	**Span Length (m)**									
	1.5	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**	**6**
75mm	13.1	9.8	7.8	5.8	3.7	2.5	1.7	1.3	D	D
100mm	17.4	13.1	10.5	8.7	6.7	4.5	3.2	2.3	1.7	1.3
125mm	21.8	16.3	13.1	10.9	9.3	7.2	5.1	3.7	2.8	2.1
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.4	6.1	4.6	3.6
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	8.8	6.6	5.1
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.0	6.9

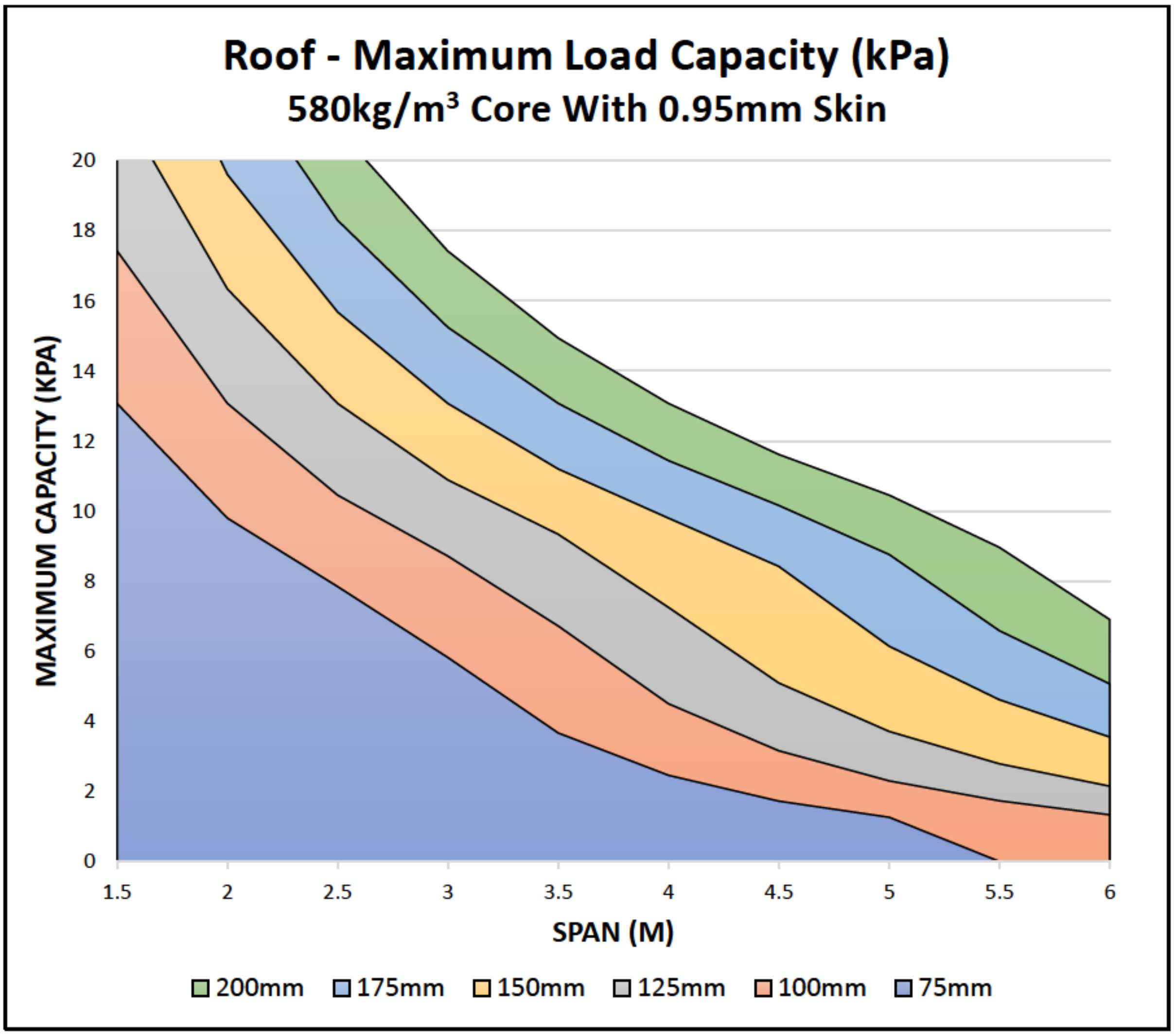


Figure L.15 - Load Capacity Graph - Roof - 580kg/m3 - 0.95mm

Table L.16 - Span Table - Roof - 580kg/m³ - 1.95mm

Roof Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm	N1 Region Max	Limiting	Deflection (D)		18.3	%		
Panel Width	600	mm	N3 Region Max		Shear (C)		81.7	%		
Core Density	580	kg/m³	N5 Region Max		Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	5.6	4.9	3.4	2.5	1.9	1.4
100mm	17.4	13.1	10.5	8.7	7.5	6.5	5.8	4.5	3.4	2.6
125mm	21.8	16.3	13.1	10.9	9.3	8.2	7.3	6.5	5.3	4.1
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.8	7.1	6.3
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	8.3	7.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.7

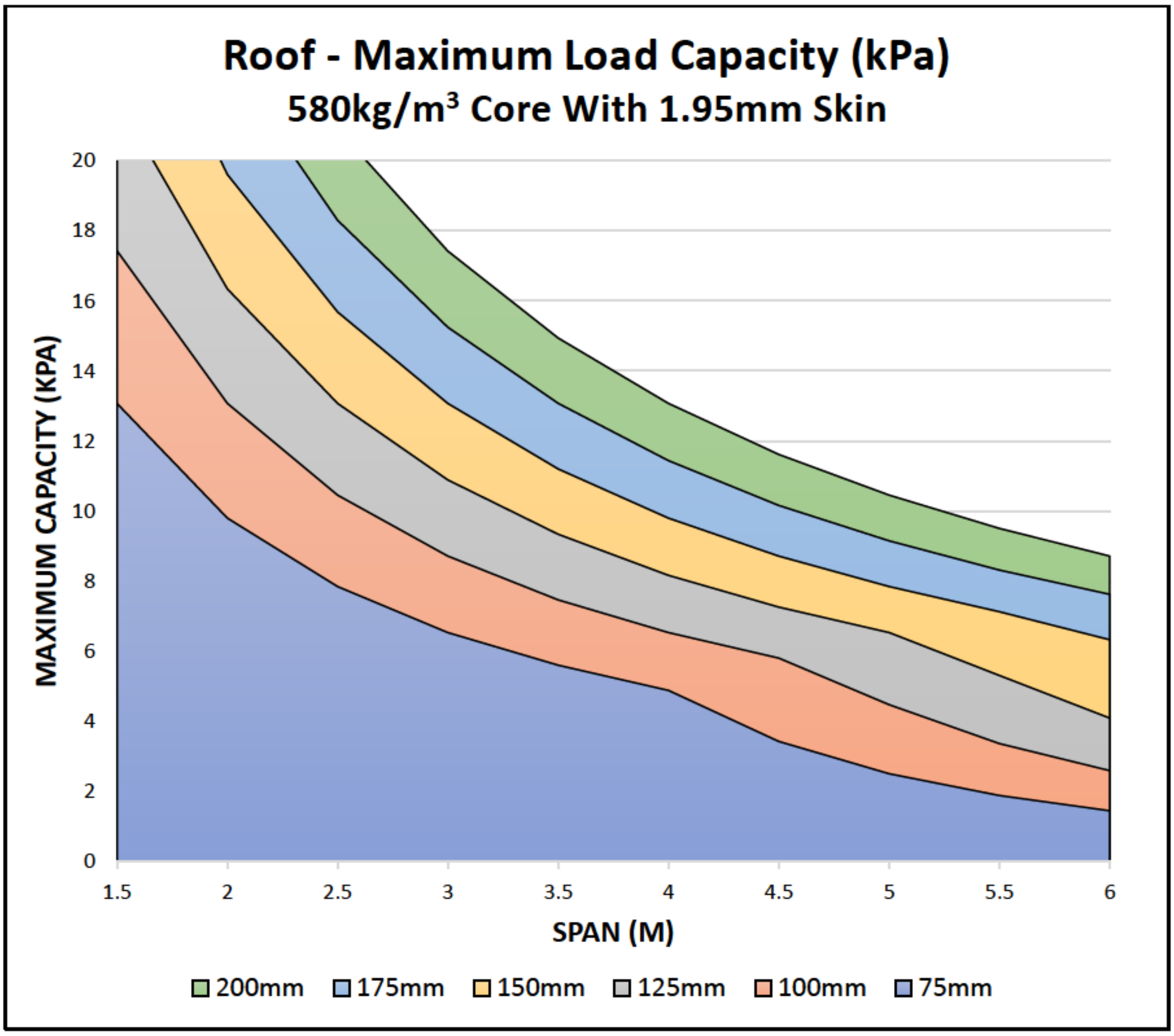


Figure L.16 - Load Capacity Graph - Roof - 580kg/m3 - 1.95mm

M Appendix M

Table M.1 - Span Table - Floor - 400kg/m³ - 0.40mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm			Limiting	Deflection (D)		60.0	%	
Panel Width	600	mm				Shear (C)		40.0	%	
Core Density	400	kg/m³				Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	C	DC	DC	DCS	DCS	DCS	DCS	DCS
100mm	10.5	7.8	6.3	3.6	DC	DC	DC	DCS	DCS	DCS
125mm	13.1	9.8	7.8	5.9	3.7	DC	DC	DC	DCS	DCS
150mm	15.7	11.8	9.4	7.8	6.7	4.7	C	DC	DC	DC
175mm	18.3	13.7	11.0	9.1	7.8	6.9	4.8	3.5	DC	DC
200mm	20.9	15.7	12.5	10.5	9.0	7.8	6.7	4.9	3.7	DC

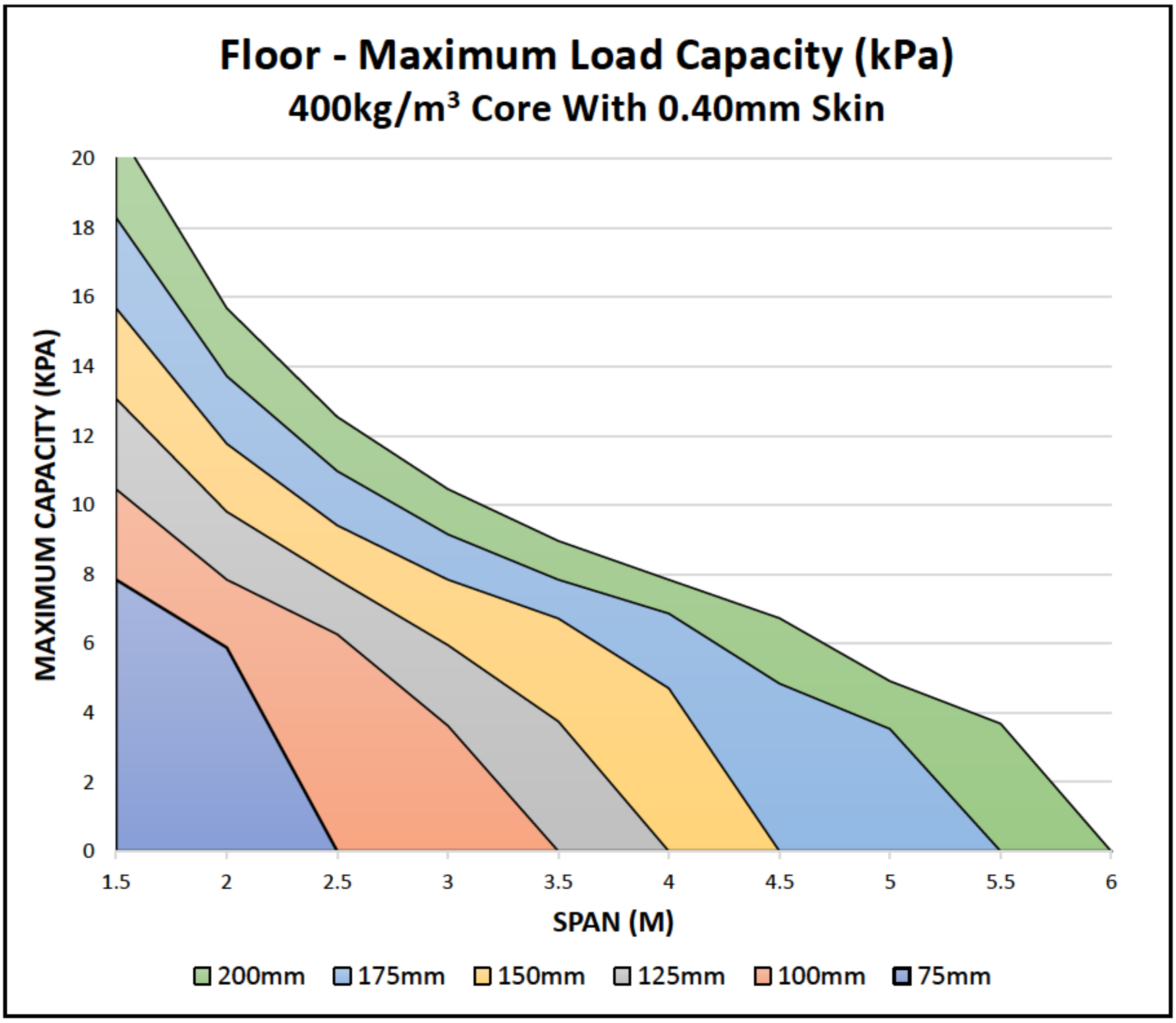


Figure M.1 - Load Capacity Graph - Floor - 400kg/m3 - 0.40mm

Table M.2 - Span Table - Floor - 400kg/m³ - 0.55mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm			Limiting	Deflection (D)		55.0	%	
Panel Width	600	mm				Shear (C)		45.0	%	
Core Density	400	kg/m³				Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	C	DC	DC	DC	DC	DCS	DCS	DCS
100mm	10.5	7.8	6.3	4.7	DC	DC	DC	DC	DCS	DCS
125mm	13.1	9.8	7.8	6.5	4.8	DC	DC	DC	DC	DCS
150mm	15.7	11.8	9.4	7.8	6.7	5.7	C	DC	DC	DC
175mm	18.3	13.7	11.0	9.1	7.8	6.9	5.8	C	DC	DC
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	5.8	4.4	C

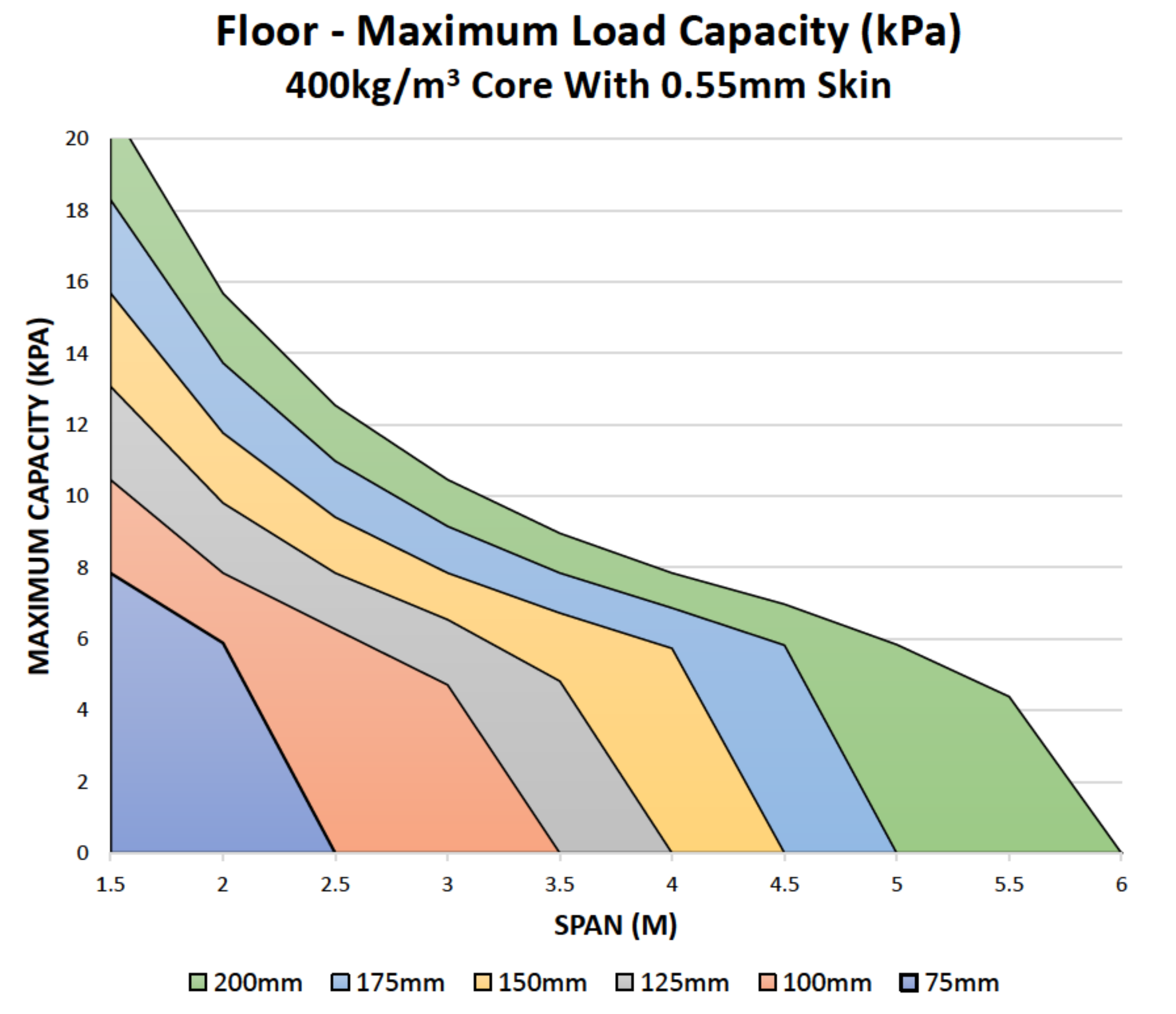


Figure M.2 - Load Capacity Graph - Floor - 400kg/m3 - 0.55mm

Table M.3 - Span Table - Floor - 400kg/m³ - 0.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm		Limiting	Deflection (D)			35.0	%	
Panel Width	600	mm			Shear (C)			65.0	%	
Core Density	400	kg/m^3			Bending (S)			0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	C	C	DC	DC	DC	DC	DC	DCS
100mm	10.5	7.8	6.3	5.2	C	DC	DC	DC	DC	DC
125mm	13.1	9.8	7.8	6.5	5.6	C	C	DC	DC	DC
150mm	15.7	11.8	9.4	7.8	6.7	5.9	C	C	DC	DC
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	C	C	C
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	5.7	C

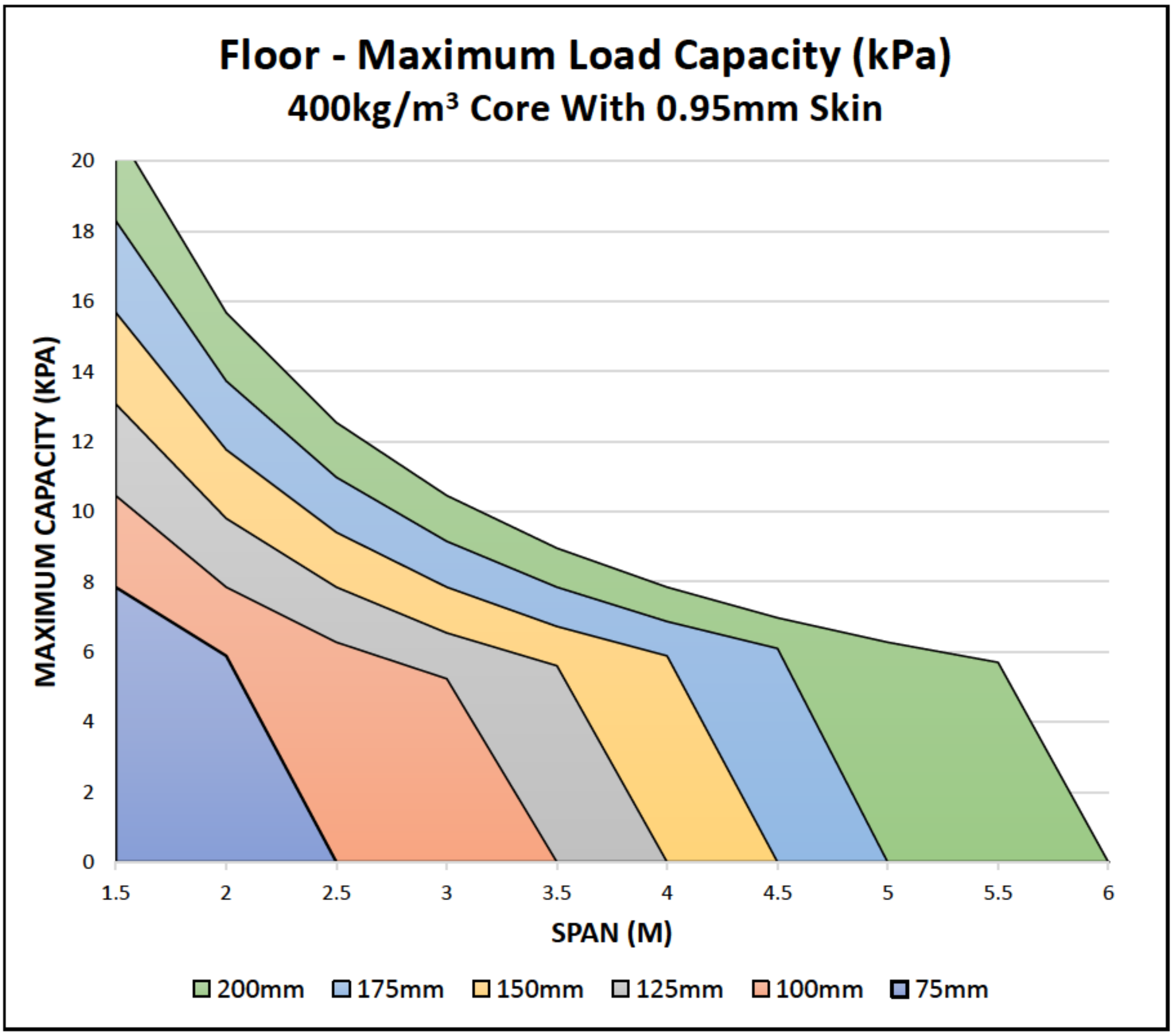


Figure M.3 - Load Capacity Graph - Floor - 400kg/m3 - 0.95mm

Table M.4 - Span Table - Floor - 400kg/m³ - 1.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm			Limiting	Deflection (D)		11.7	%	
Panel Width	600	mm				Shear (C)		88.3	%	
Core Density	400	kg/m³				Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	7.8	5.9	C	C	C	C	DC	DC	DC	DC
100mm	10.5	7.8	6.3	C	C	C	C	DC	DC	DC
125mm	13.1	9.8	7.8	6.5	5.6	C	C	C	C	DC
150mm	15.7	11.8	9.4	7.8	6.7	5.9	C	C	C	C
175mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	C	C	C
200mm	20.9	15.7	12.5	10.5	9.0	7.8	7.0	6.3	C	C

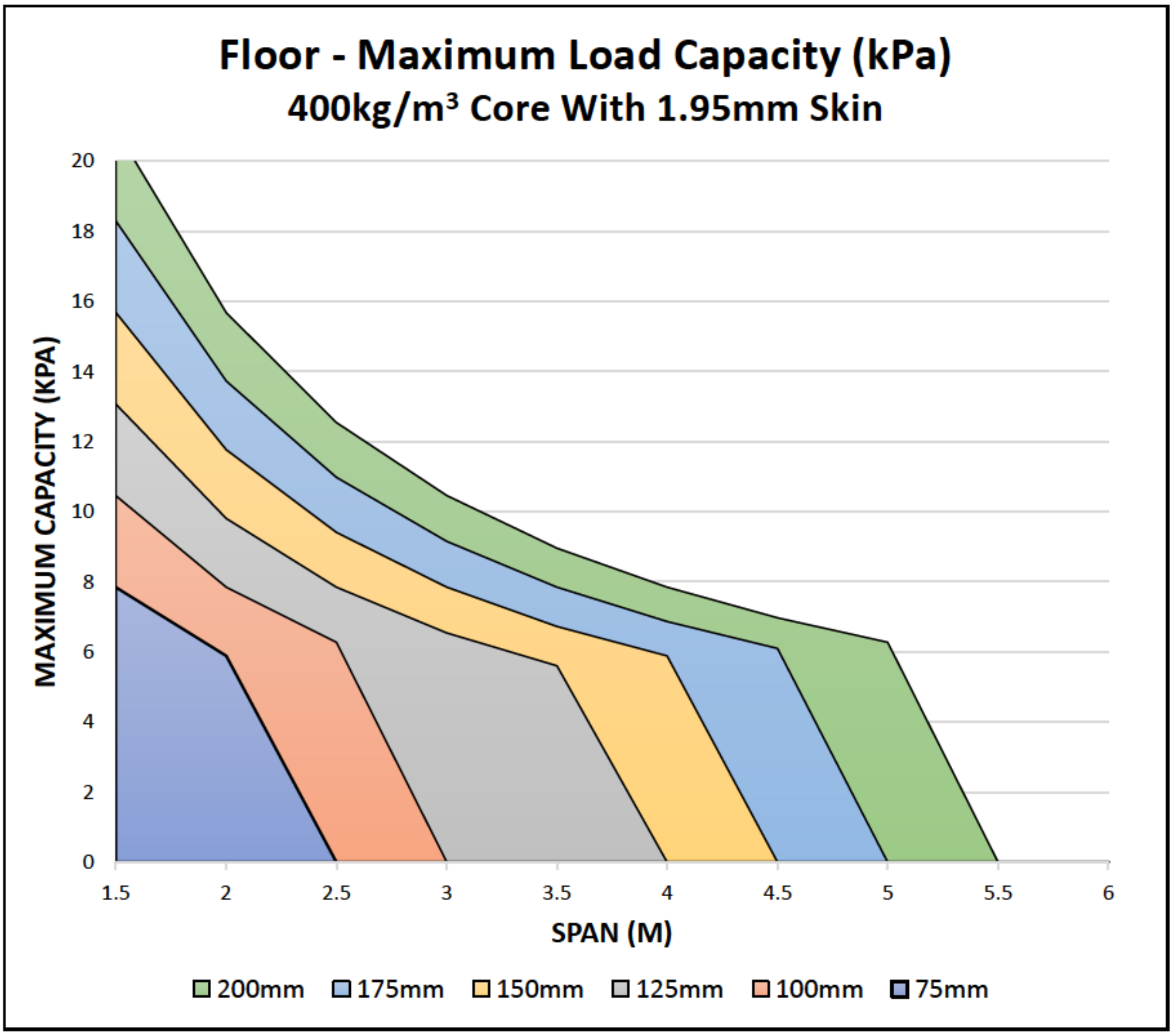


Figure M.4 - Load Capacity Graph - Floor - 400kg/m3 - 1.95mm

Table M.5 - Span Table - Floor - 510kg/m³ - 0.40mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.4	mm		Limiting	Deflection (D)		65.0	%		
Panel Width	600	mm			Shear (C)		35.0	%		
Core Density	510	kg/m^3			Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.8	3.5	DC	DC	DC	DCS	DCS	DCS	DCS
100mm	12.2	9.1	6.6	3.8	D	DC	DC	DCS	DCS	DCS
125mm	15.2	11.4	9.1	6.3	4.0	D	DC	DC	DCS	DCS
150mm	18.3	13.7	11.0	9.1	7.4	5.0	3.5	DC	DC	DC
175mm	21.3	16.0	12.8	10.7	9.1	7.3	5.1	3.8	D	DC
200mm	24.4	18.3	14.6	12.2	10.5	9.1	7.2	5.2	3.9	D

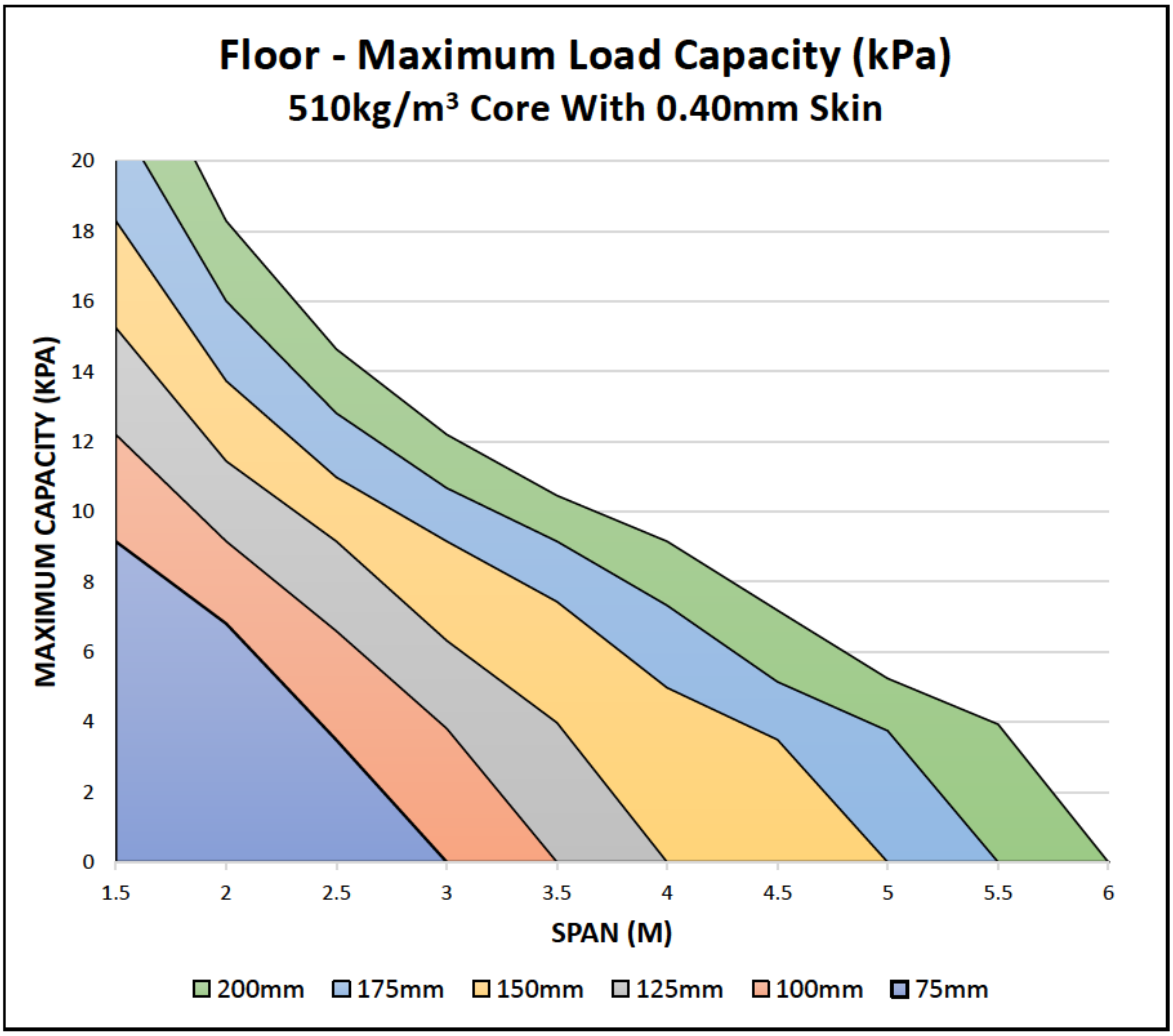


Figure M.5 - Load Capacity Graph - Floor - 510kg/m3 - 0.40mm

Table M.6 - Span Table - Floor - 510kg/m³ - 0.55mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm		Limiting	Deflection (D)			55.0	%	
Panel Width	600	mm			Shear (C)			45.0	%	
Core Density	510	kg/m³			Bending (S)			0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	4.5	DC	DC	DC	DC	DCS	DCS	DCS
100mm	12.2	9.1	7.3	4.9	D	DC	DC	DC	DCS	DCS
125mm	15.2	11.4	9.1	7.6	5.0	3.4	DC	DC	DC	DCS
150mm	18.3	13.7	11.0	9.1	7.8	6.0	4.2	DC	DC	DC
175mm	21.3	16.0	12.8	10.7	9.1	8.0	6.1	4.5	3.4	DC
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	6.2	4.6	3.6

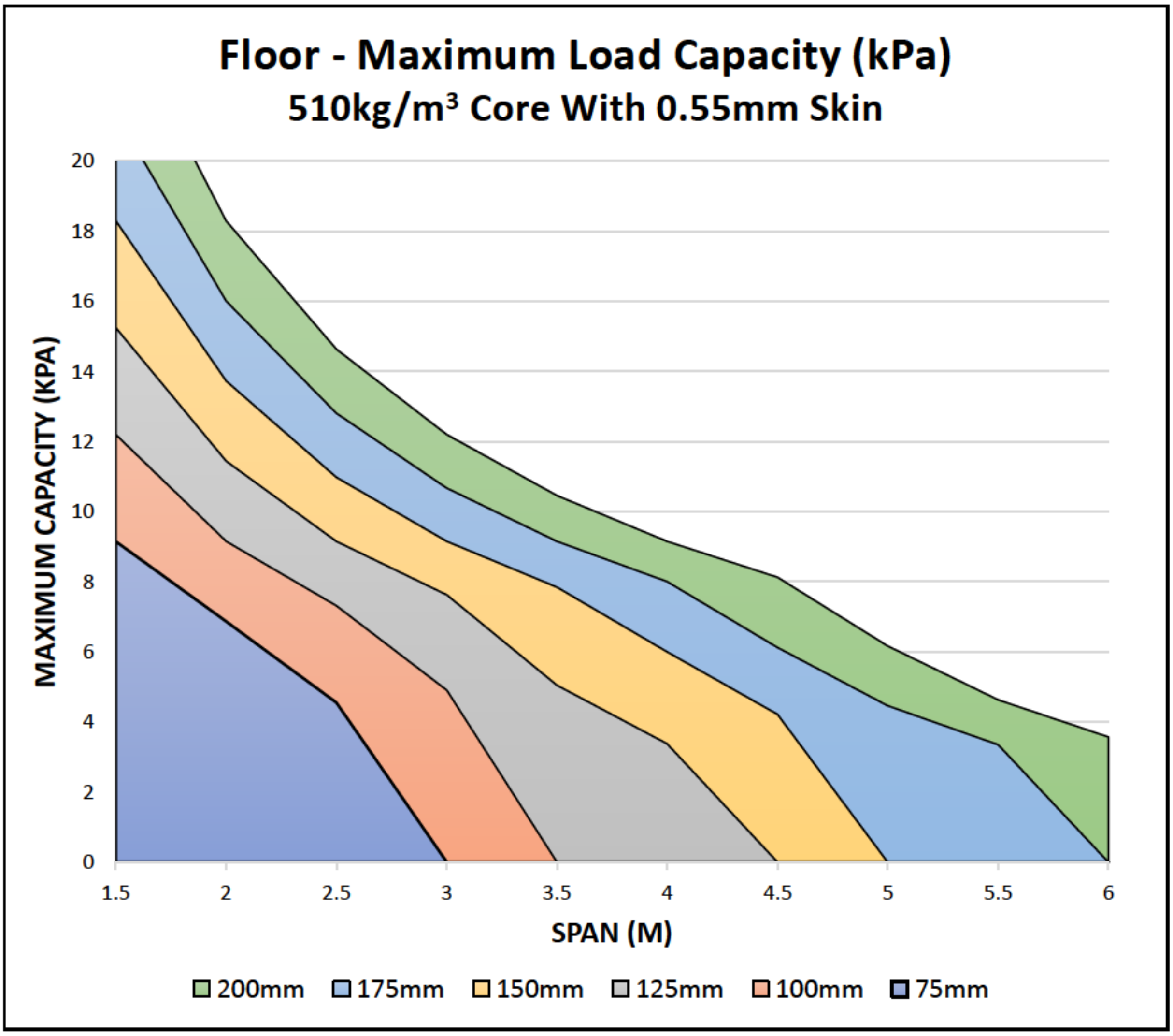


Figure M.6 - Load Capacity Graph - Floor - 510kg/m3 - 0.55mm

Table M.7 - Span Table - Floor - 510kg/m³ - 0.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm		Limiting	Deflection (D)		43.3	%		
Panel Width	600	mm			Shear (C)		56.7	%		
Core Density	510	kg/m³			Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	9.1	6.9	5.5	C	DC	DC	DC	DC	DC	DCS
100mm	12.2	9.1	7.3	6.1	C	DC	DC	DC	DC	DC
125mm	15.2	11.4	9.1	7.6	6.5	5.3	C	DC	DC	DC
150mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	C	C	DC
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.4	4.8	C
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.5	5.0

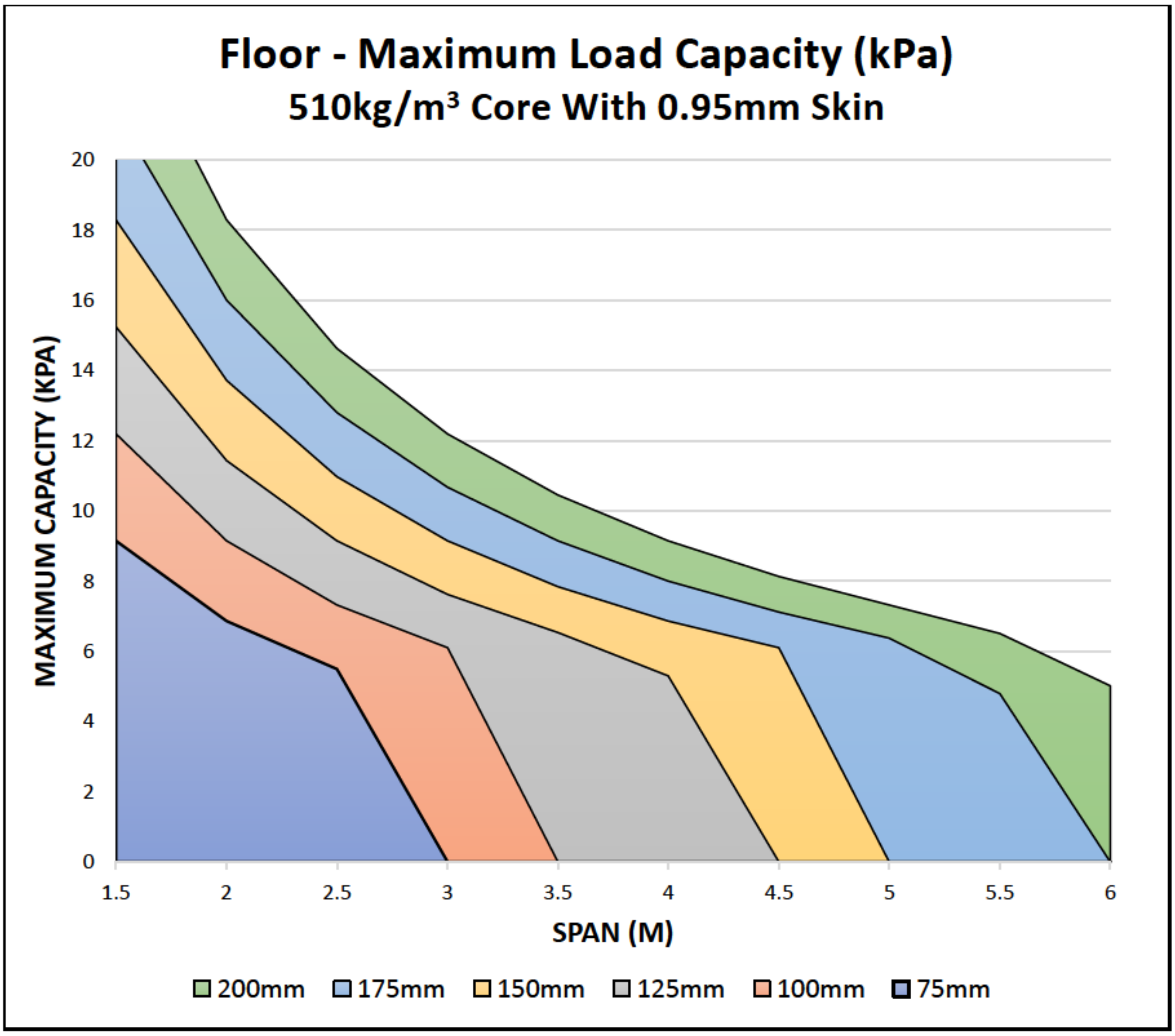


Figure M.7 - Load Capacity Graph - Floor - 510kg/m3 - 0.95mm

Table M.8 - Span Table - Floor - 510kg/m³ - 1.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm		Limiting	Deflection (D)		15.0	%		
Panel Width	600	mm			Shear (C)		85.0	%		
Core Density	510	kg/m^3			Bending (S)		0.0	%		
Core Thickness	**Span Length (m)**									
	1.5	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**	**6**
75mm	9.1	6.9	5.5	C	C	C	DC	DC	DC	DC
100mm	12.2	9.1	7.3	6.1	C	C	C	DC	DC	DC
125mm	15.2	11.4	9.1	7.6	6.5	5.7	C	C	C	DC
150mm	18.3	13.7	11.0	9.1	7.8	6.9	6.1	C	C	C
175mm	21.3	16.0	12.8	10.7	9.1	8.0	7.1	6.4	C	C
200mm	24.4	18.3	14.6	12.2	10.5	9.1	8.1	7.3	6.7	C

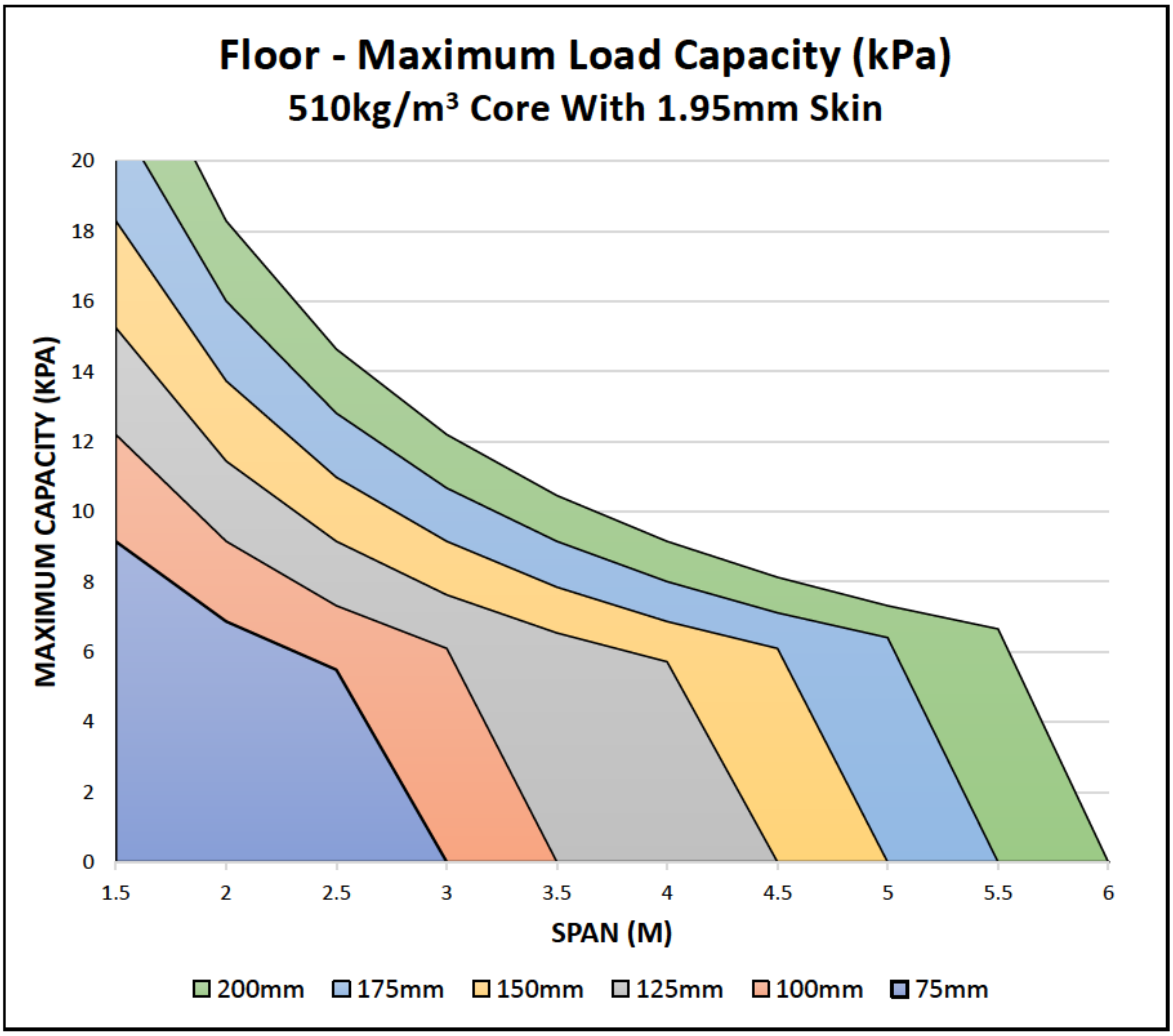


Figure M.8 - Load Capacity Graph - Floor - 510kg/m3 - 1.95mm

Table M.9 - Span Table - Floor - 550kg/m³ - 0.40mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm			Limiting	Deflection (D)			71.7	%
Panel Width	600	mm				Shear (C)			28.3	%
Core Density	550	kg/m^3				Bending (S)			0.0	%
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	6.9	3.6	D	D	DC	DCS	DCS	DCS	DCS
100mm	17.4	13.1	6.8	3.9	D	D	D	DCS	DCS	DCS
125mm	21.8	16.3	11.2	6.5	4.1	D	D	D	DS	DS
150mm	26.1	19.6	15.7	12.1	7.6	5.1	3.6	D	D	D
175mm	30.5	22.9	18.3	15.2	11.3	7.5	5.3	3.9	D	D
200mm	34.8	26.1	20.9	17.4	14.9	10.5	7.4	5.4	4.1	D

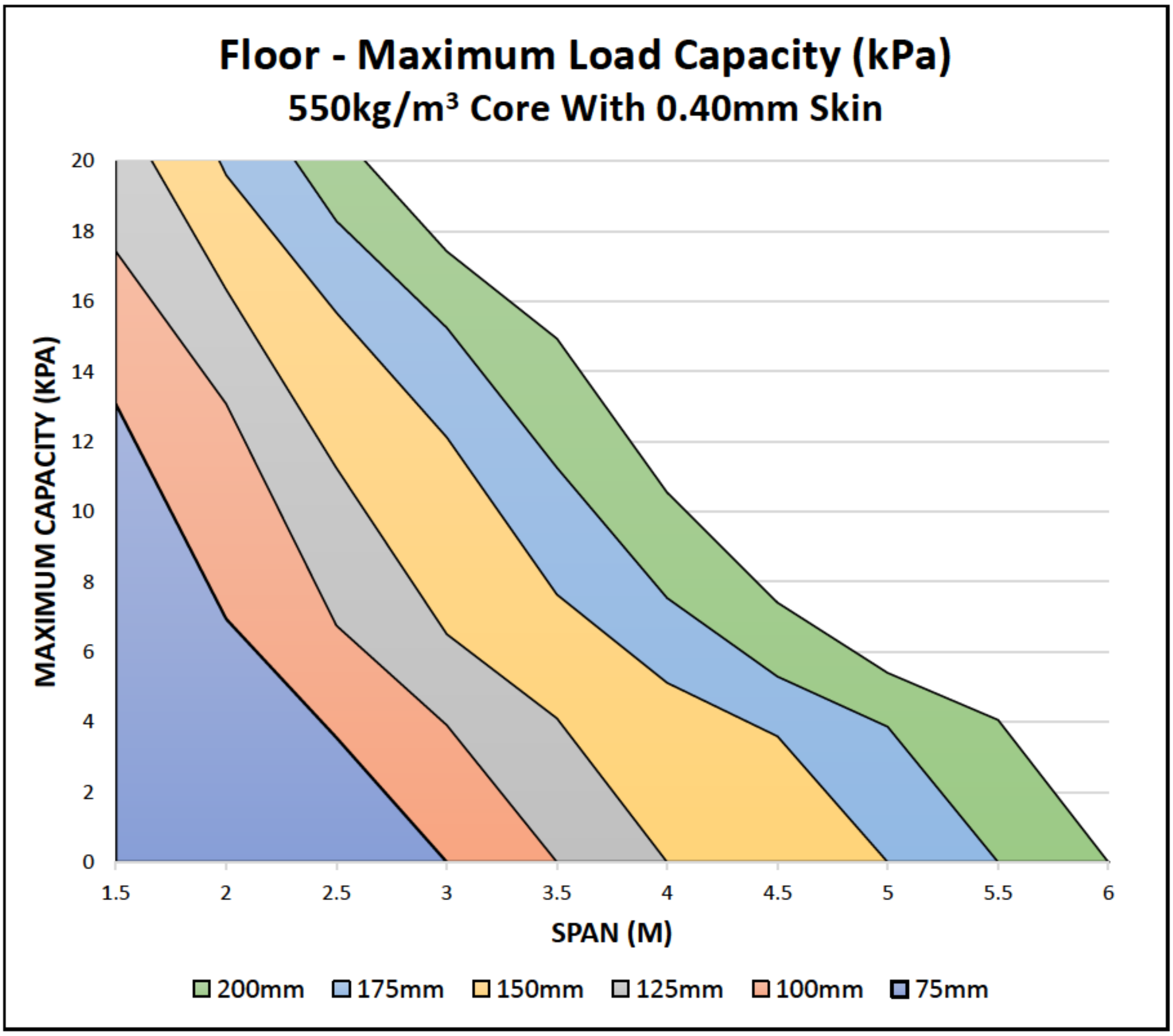


Figure M.9 - Load Capacity Graph - Floor - 550kg/m3 - 0.40mm

Table M.10 - Span Table - Floor - 550kg/m³ - 0.55mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55 mm			Limiting	Deflection (D)			66.7 %		
Panel Width	600 mm				Shear (C)			33.3 %		
Core Density	550 kg/m³				Bending (S)			0.0 %		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.0	4.6	D	D	DC	DC	DCS	DCS	DCS
100mm	17.4	13.1	8.6	5.0	D	D	D	DC	DCS	DCS
125mm	21.8	16.3	13.1	8.2	5.2	3.5	D	D	D	DC
150mm	26.1	19.6	15.7	13.1	9.2	6.1	4.3	D	D	D
175mm	30.5	22.9	18.3	15.2	13.1	8.9	6.3	4.6	3.4	D
200mm	34.8	26.1	20.9	17.4	14.9	12.4	8.7	6.3	4.8	3.7

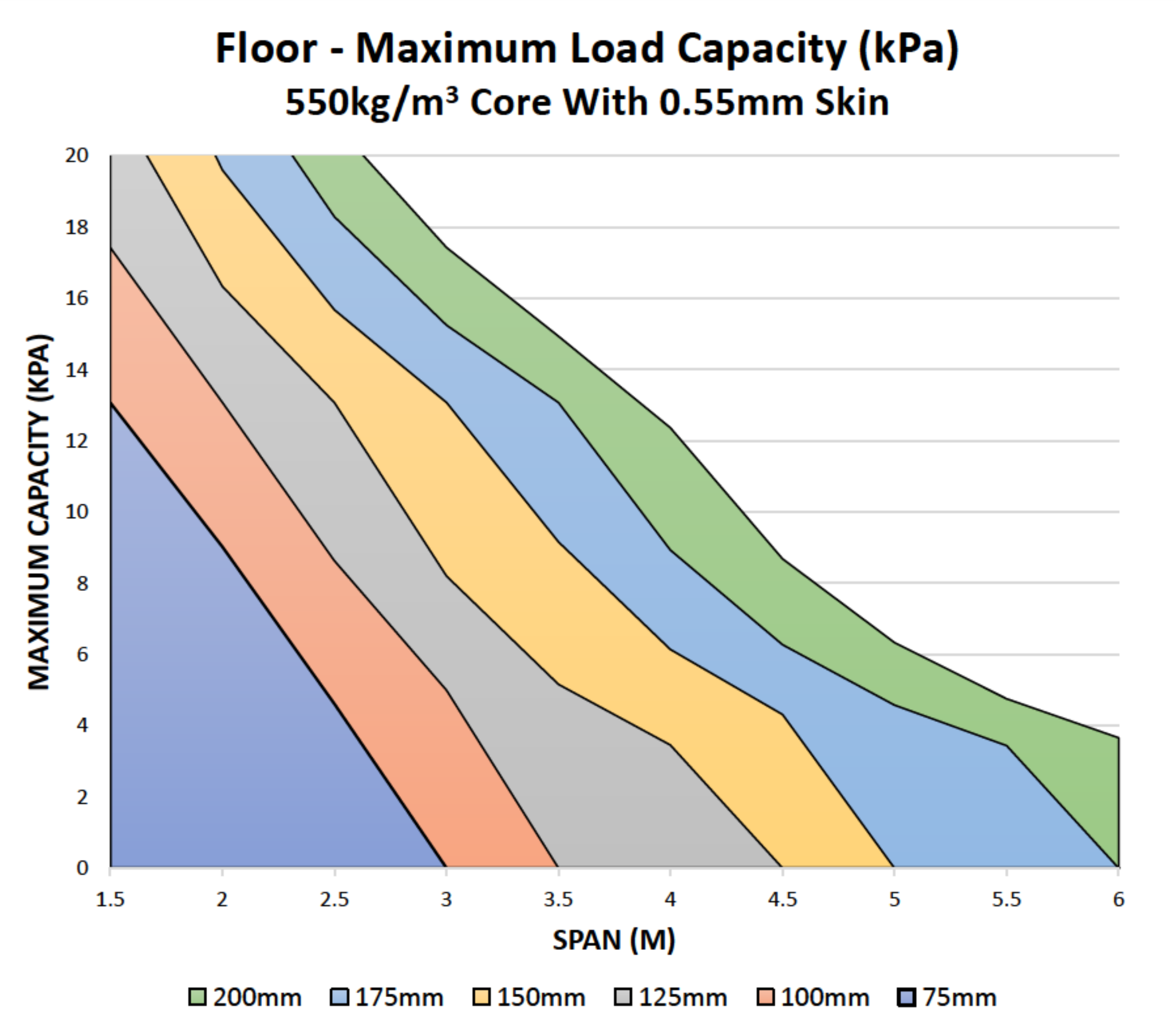


Figure M.10 - Load Capacity Graph - Floor - 550kg/m3 - 0.55mm

Table M.11 - Span Table - Floor - 550kg/m³ - 0.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm		Limiting	Deflection (D)		55.0	%		
Panel Width	600	mm			Shear (C)		45.0	%		
Core Density	550	kg/m^3			Bending (S)		0.0	%		
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.5	4.3	D	DC	DC	DC	DC	DCS
100mm	17.4	13.1	10.5	7.9	5.0	3.3	D	DC	DC	DC
125mm	21.8	16.3	13.1	10.9	8.0	5.4	3.8	D	D	DC
150mm	26.1	19.6	15.7	13.1	11.2	8.9	6.2	4.6	3.4	D
175mm	30.5	22.9	18.3	15.2	13.1	11.4	8.9	6.5	4.9	3.8
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	8.8	6.6	5.1

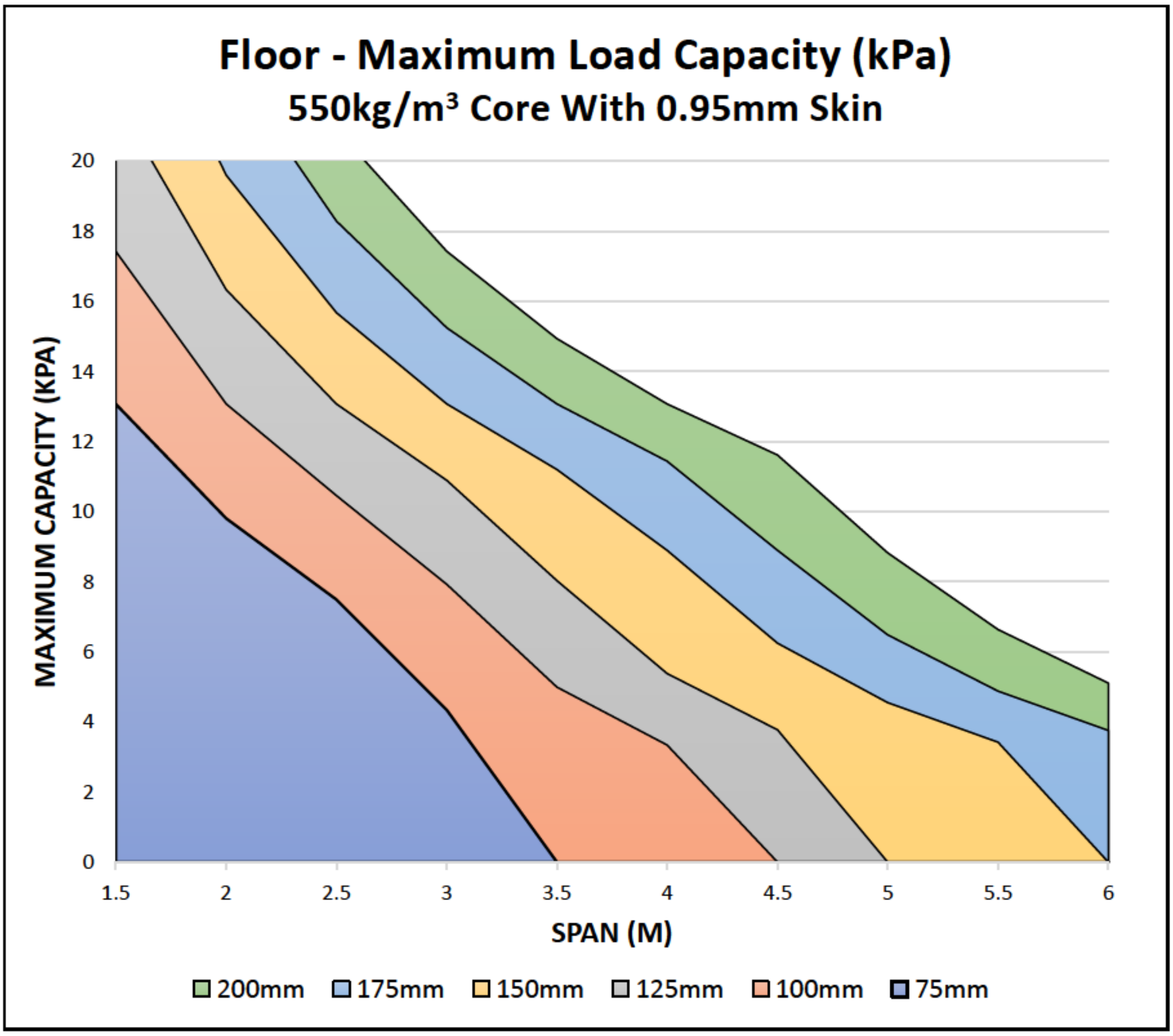


Figure M.11 - Load Capacity Graph - Floor - 550kg/m3 - 0.95mm

Table M.12 - Span Table - Floor - 550kg/m³ - 1.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm			Limiting	Deflection (D)		30	%	
Panel Width	600	mm				Shear (C)		70	%	
Core Density	550	kg/m^3				Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	5.4	C	DC	DC	DC	DC
100mm	17.4	13.1	10.5	8.7	7.5	6.5	4.6	C	DC	DC
125mm	21.8	16.3	13.1	10.9	9.3	8.2	7.2	5.3	4.0	DC
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.8	6.1	4.7
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	8.3	6.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.7

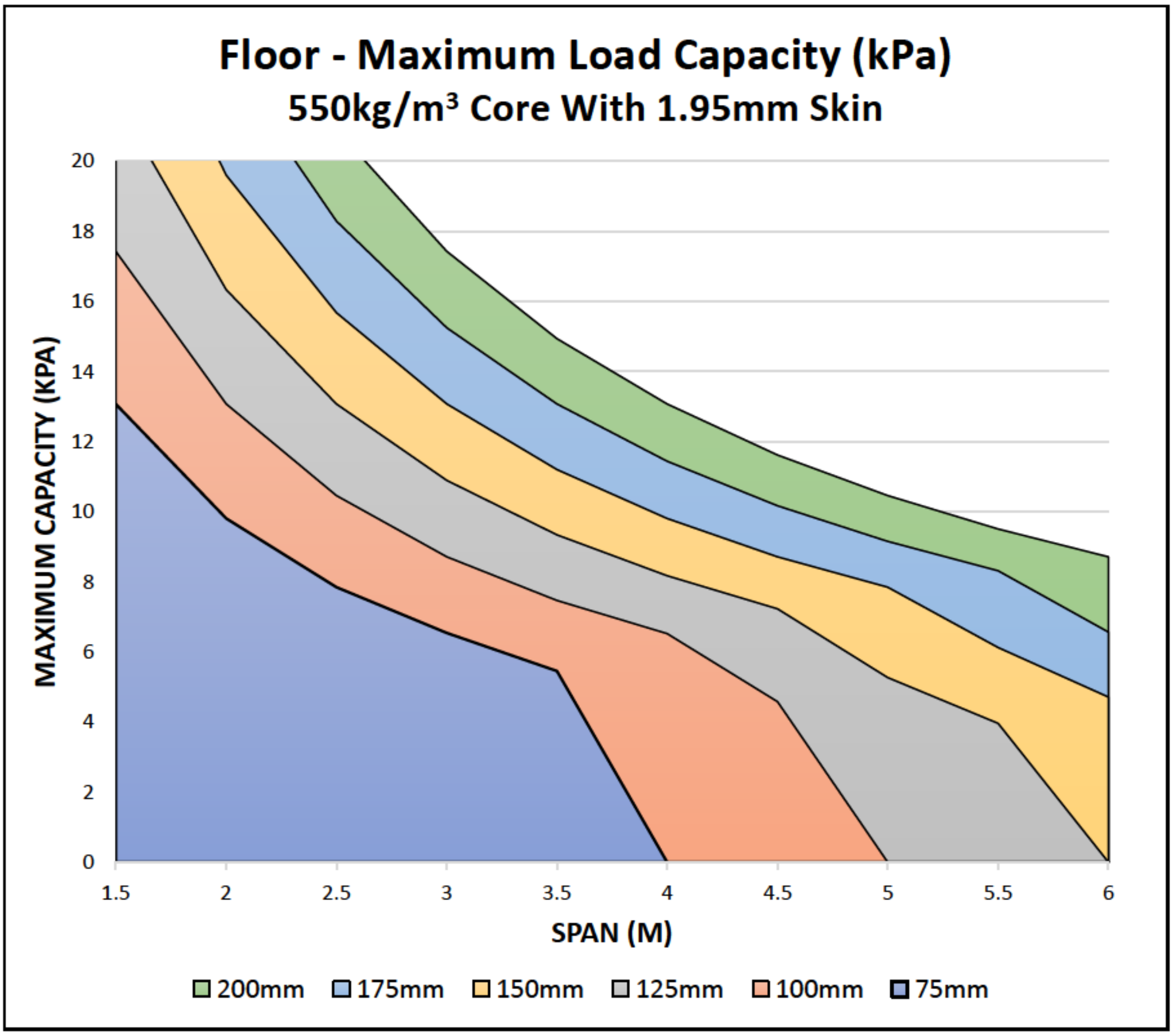


Figure M.12 - Load Capacity Graph - Floor - 550kg/m3 - 1.95mm

Table M.13 - Span Table - Floor - 580kg/m³ - 0.40mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.40	mm			Limiting	Deflection (D)		71.7	%	
Panel Width	600	mm				Shear (C)		28.3	%	
Core Density	580	kg/m^3				Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	7.0	3.6	D	D	DC	DCS	DCS	DCS	DCS
100mm	17.4	13.1	6.9	4.0	D	D	D	DCS	DCS	DCS
125mm	21.8	16.3	11.5	6.6	4.2	D	D	D	DS	DCS
150mm	26.1	19.6	15.7	12.4	7.8	5.2	3.7	D	D	D
175mm	30.5	22.9	18.3	15.2	11.5	7.7	5.4	3.9	D	D
200mm	34.8	26.1	20.9	17.4	14.9	10.8	7.6	5.5	4.1	D

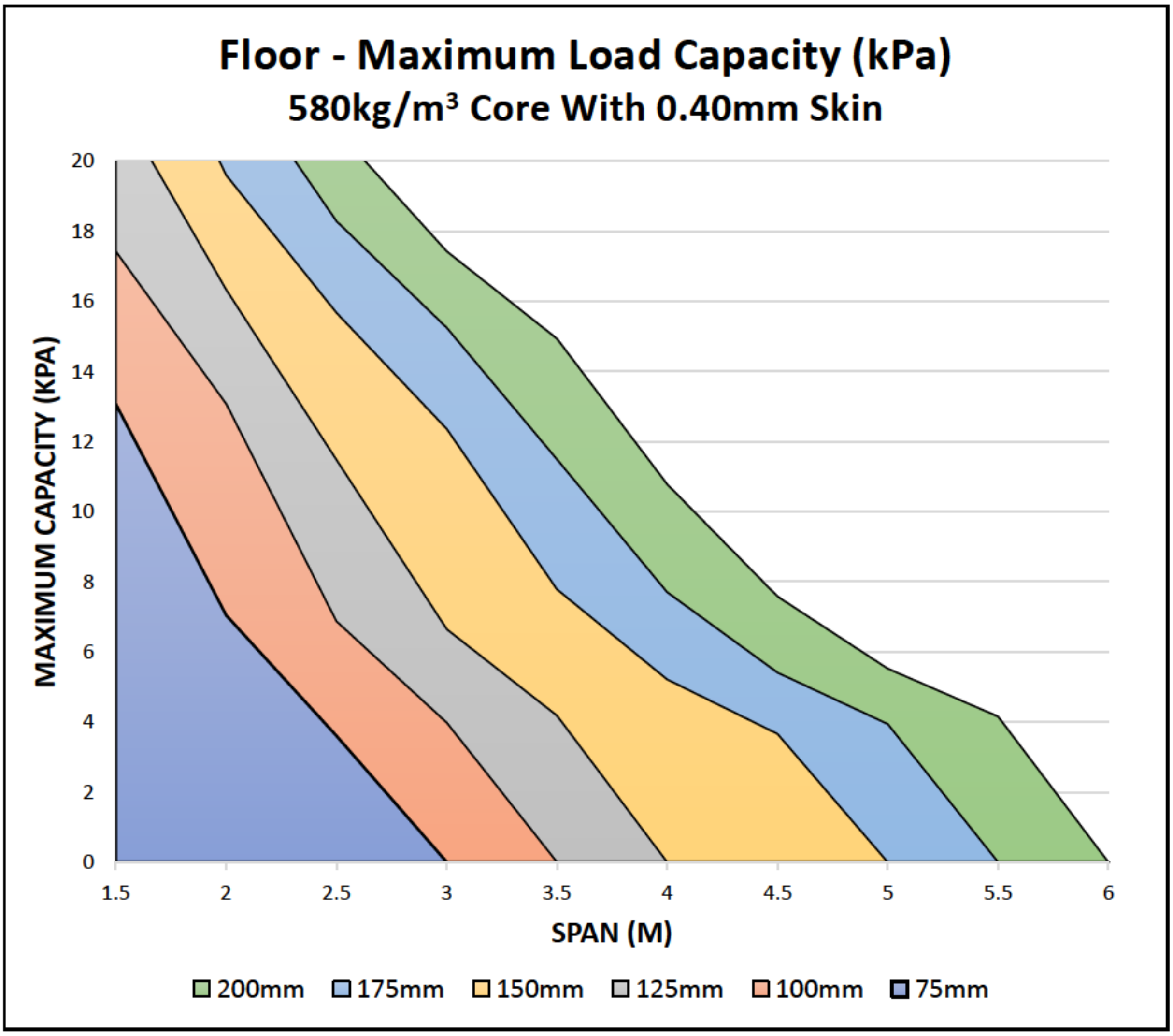


Figure M.13 - Load Capacity Graph - Floor - 580kg/m3 - 0.40mm

Table M.14 - Span Table - Floor - 580kg/m³ - 0.55mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm			Limiting	Deflection (D)			66.7	%
Panel Width	600	mm				Shear (C)			33.3	%
Core Density	580	kg/m³				Bending (S)			0.0	%
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.1	4.7	D	D	DC	DC	DCS	DCS	DCS
100mm	17.4	13.1	8.8	5.1	D	D	D	DC	DCS	DCS
125mm	21.8	16.3	13.1	8.3	5.2	3.5	D	D	D	DC
150mm	26.1	19.6	15.7	13.1	9.3	6.2	4.4	D	D	D
175mm	30.5	22.9	18.3	15.2	13.1	9.1	6.4	4.7	3.5	D
200mm	34.8	26.1	20.9	17.4	14.9	12.6	8.9	6.5	4.8	3.7

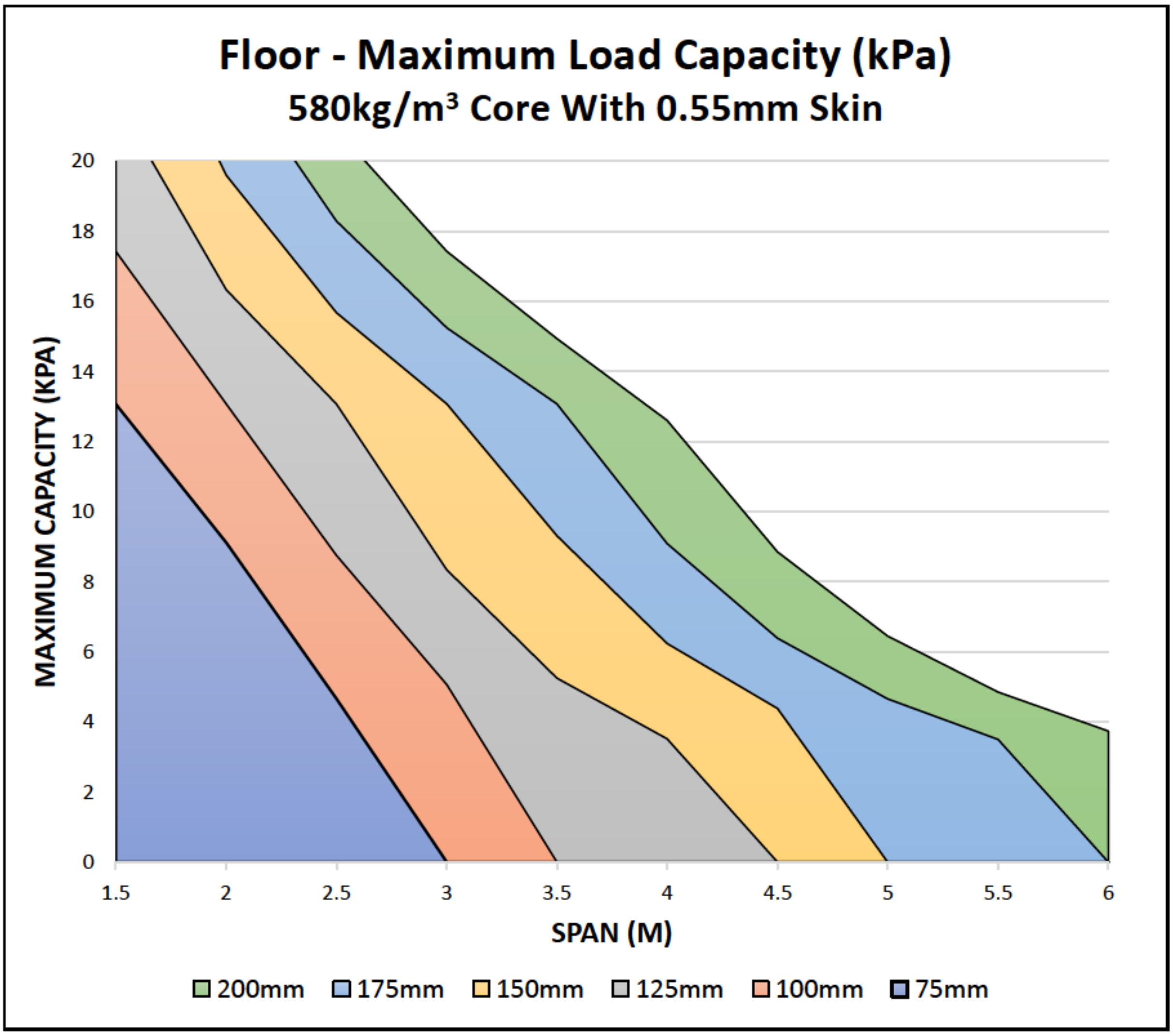


Figure M.14 - Load Capacity Graph - Floor - 580kg/m3 - 0.55mm

Table M.15 - Span Table - Floor - 580kg/m³ - 0.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm		Limiting	Deflection (D)			55.0	%	
Panel Width	600	mm			Shear (C)			45.0	%	
Core Density	580	kg/m³			Bending (S)			0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.5	4.4	D	D	D	D	D	D
100mm	17.4	13.1	10.5	8.0	5.0	3.4	D	D	D	D
125mm	21.8	16.3	13.1	10.9	8.1	5.4	3.8	D	D	D
150mm	26.1	19.6	15.7	13.1	11.2	9.0	6.3	4.6	3.5	D
175mm	30.5	22.9	18.3	15.2	13.1	11.4	9.0	6.6	4.9	3.8
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	8.9	6.7	5.2

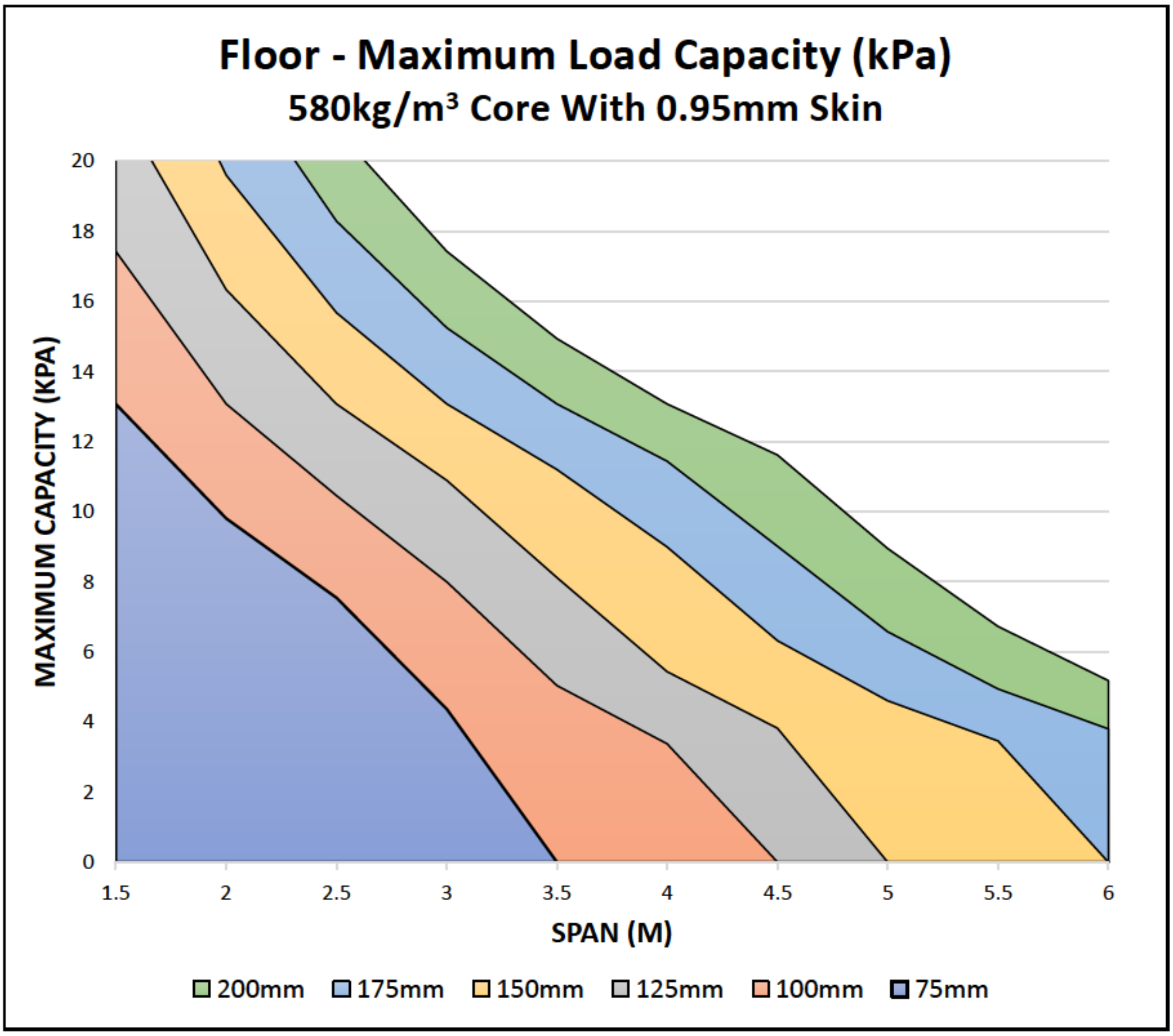


Figure M.15 - Load Capacity Graph - Floor - 580kg/m3 - 0.95mm

Table M.16 - Span Table - Floor - 580kg/m³ - 1.95mm

Floor Application Maximum Load Capacity (kPa) Table Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm			Limiting	Deflection (D)		26.7	%	
Panel Width	600	mm				Shear (C)		73.3	%	
Core Density	580	kg/m^3				Bending (S)		0.0	%	
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	9.8	7.8	6.5	5.5	C	DC	DC	DC	DC
100mm	17.4	13.1	10.5	8.7	7.5	6.5	4.6	C	DC	DC
125mm	21.8	16.3	13.1	10.9	9.3	8.2	7.3	5.3	4.0	DC
150mm	26.1	19.6	15.7	13.1	11.2	9.8	8.7	7.8	6.2	4.7
175mm	30.5	22.9	18.3	15.2	13.1	11.4	10.2	9.1	8.3	6.6
200mm	34.8	26.1	20.9	17.4	14.9	13.1	11.6	10.5	9.5	8.7

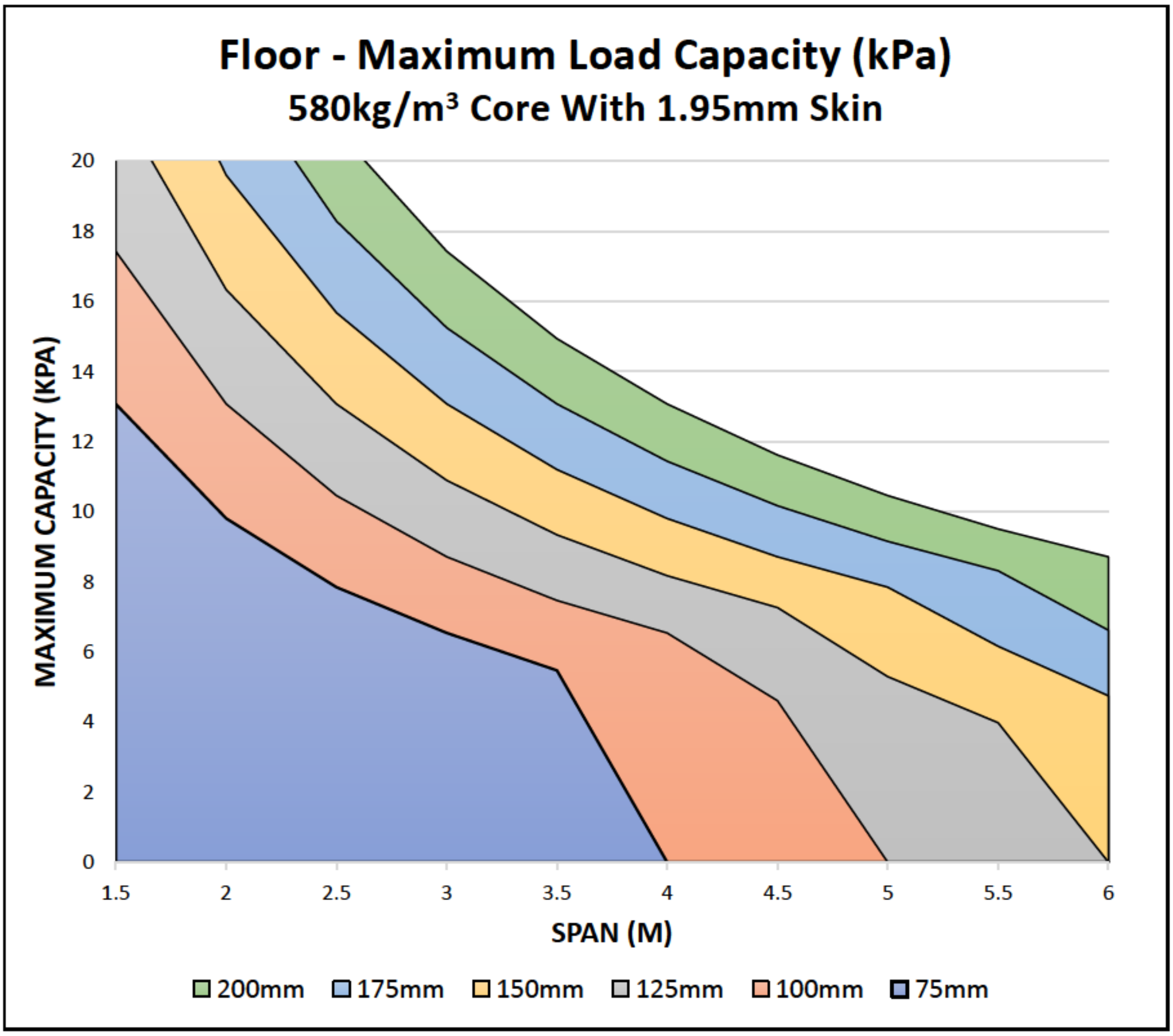


Figure M.16 - Load Capacity Graph - Floor - 580kg/m3 - 1.95mm

N Appendix N

Table N.1 - Span Table - Formwork - 400kg/m3 - 0.40mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.4 mm										
Panel Width 600 mm										
Core Density 400 kg/m³										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	C	C	C	C	CF	CF	CF	CF	CFD	CFD
100mm	C	C	C	C	C	C	CF	CF	CF	CFD
125mm	13.1	C	C	C	C	C	C	CF	CF	CF
150mm	15.7	11.8	C	C	C	C	C	C	C	CF
175mm	18.3	13.7	C	C	C	C	C	C	C	C
200mm	20.9	15.7	12.5	C	C	C	C	C	C	C

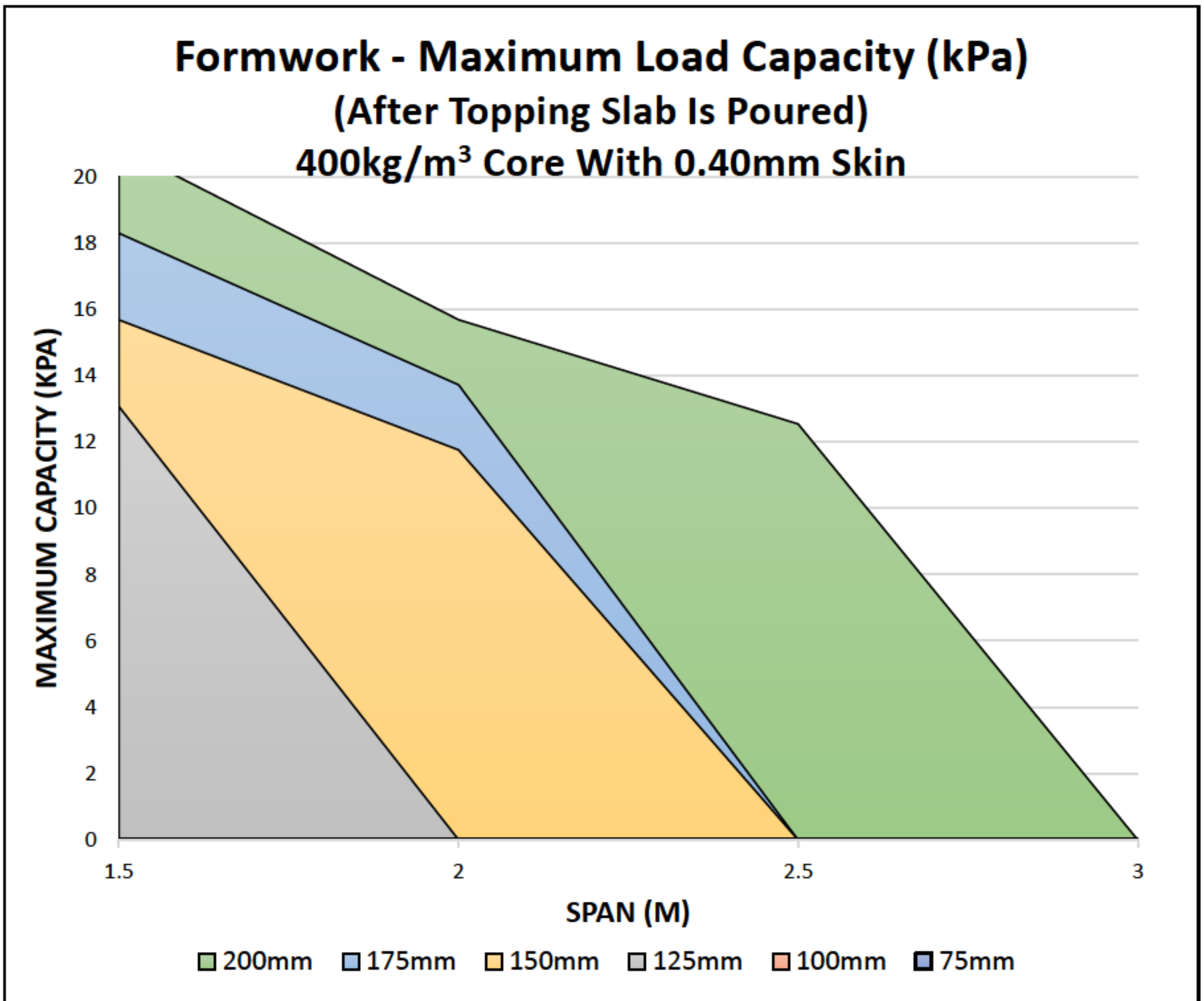


Figure N.1 - Load Capacity Graph - Formwork - 400kg/m3 - 0.40mm

Table 10.2 - Span Table - Formwork - 400kg/m3 - 0.55mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm								
Panel Width	600	mm								
Core Density	400	kg/m^3								
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	C	C	C	C	C	CF	CF	CF	CF	CFD
100mm	C	C	C	C	C	C	CF	CF	CF	CF
125mm	13.1	C	C	C	C	C	C	C	CF	CF
150mm	15.7	11.8	C	C	C	C	C	C	C	C
175mm	18.3	13.7	C	C	C	C	C	C	C	C
200mm	20.9	15.7	12.5	C	C	C	C	C	C	C

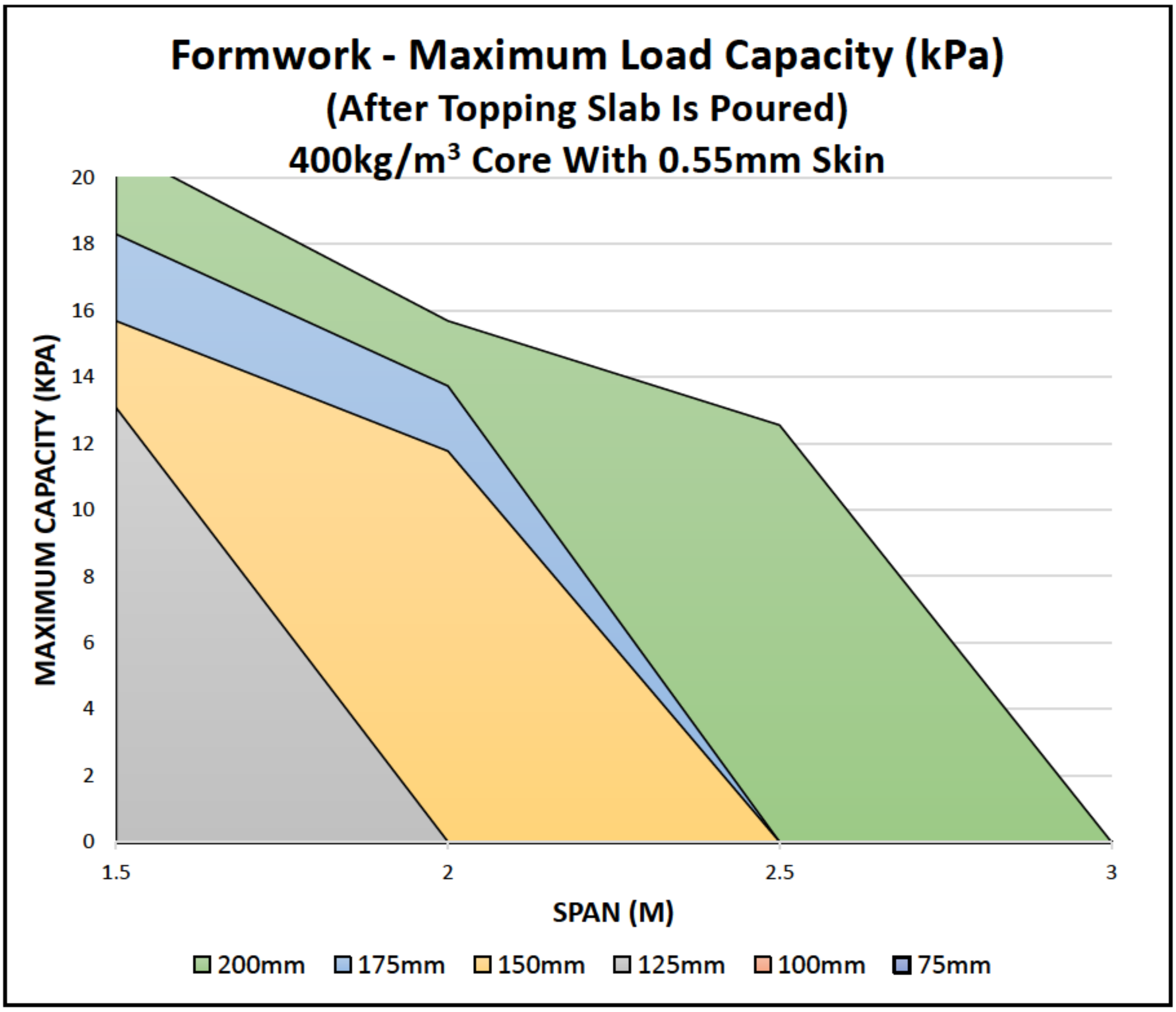


Figure N.2 - Load Capacity Graph - Formwork - 400kg/m3 - 0.55mm

Table N.3 - Span Table - Formwork - 400kg/m3 - 0.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95 mm									
Panel Width	600 mm									
Core Density	400 kg/m^3									
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	C	C	C	C	C	C	CF	CF	CF	CF
100mm	C	C	C	C	C	C	C	C	CF	CF
125mm	13.1	C	C	C	C	C	C	C	C	C
150mm	15.7	11.8	C	C	C	C	C	C	C	C
175mm	18.3	13.7	C	C	C	C	C	C	C	C
200mm	20.9	15.7	12.5	C	C	C	C	C	C	C

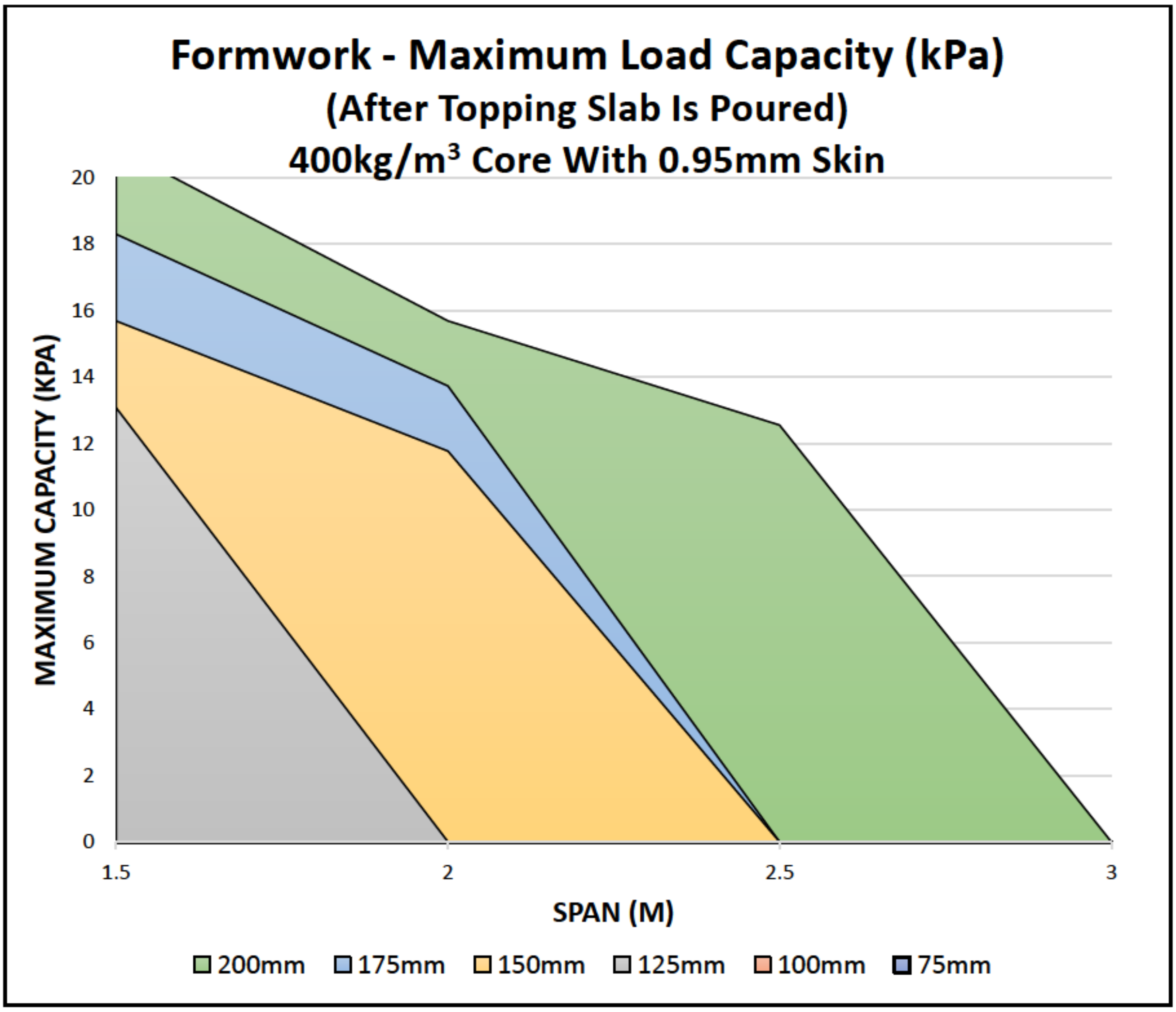


Figure N.3 - Load Capacity Graph - Formwork - 400kg/m3 - 0.95mm

Table N.4 - Span Table - Formwork - 400kg/m3 - 1.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	1.95	mm								
Panel Width	600	mm								
Core Density	400	kg/m^3								
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	C	C	C	C	C	C	C	C	CF	CF
100mm	C	C	C	C	C	C	C	C	C	C
125mm	13.1	C	C	C	C	C	C	C	C	C
150mm	15.7	11.8	C	C	C	C	C	C	C	C
175mm	18.3	13.7	C	C	C	C	C	C	C	C
200mm	20.9	15.7	12.5	C	C	C	C	C	C	C

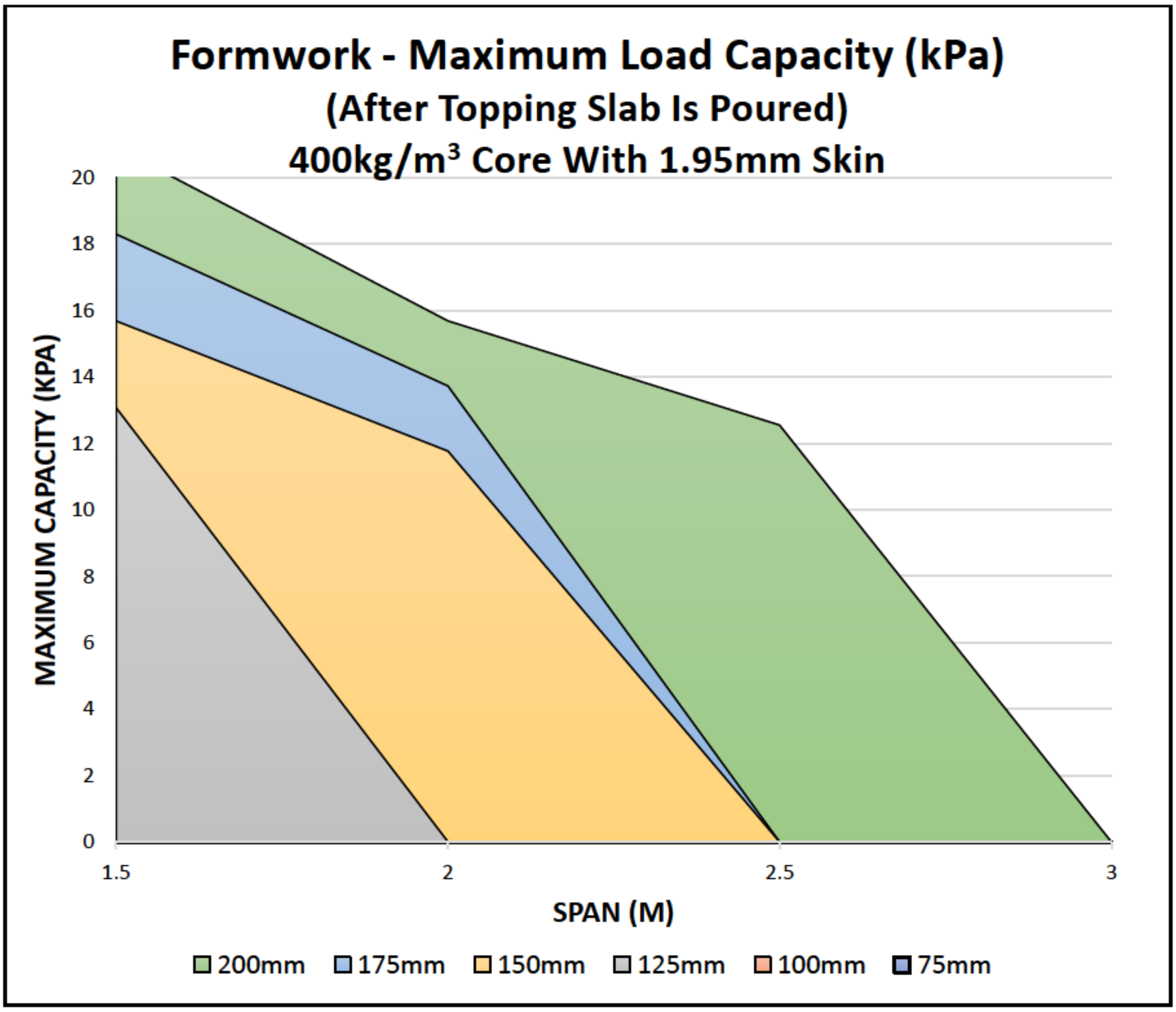


Figure N.4 - Load Capacity Graph - Formwork - 400kg/m3 - 1.95mm

Table N.5 - Span Table - Formwork - 510kg/m3 - 0.40mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.4 mm										
Panel Width 600 mm										
Core Density 510 kg/m³										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	C	C	C	C	CF	CF	CF	CF	CFD	CFD
100mm	12.2	C	C	C	C	C	CF	CF	CF	CF
125mm	15.2	11.4	C	C	C	C	C	CF	CF	CF
150mm	18.3	13.7	C	C	C	C	C	C	C	CF
175mm	21.3	16.0	12.8	C	C	C	C	C	C	C
200mm	24.4	18.3	14.6	12.2	C	C	C	C	C	C

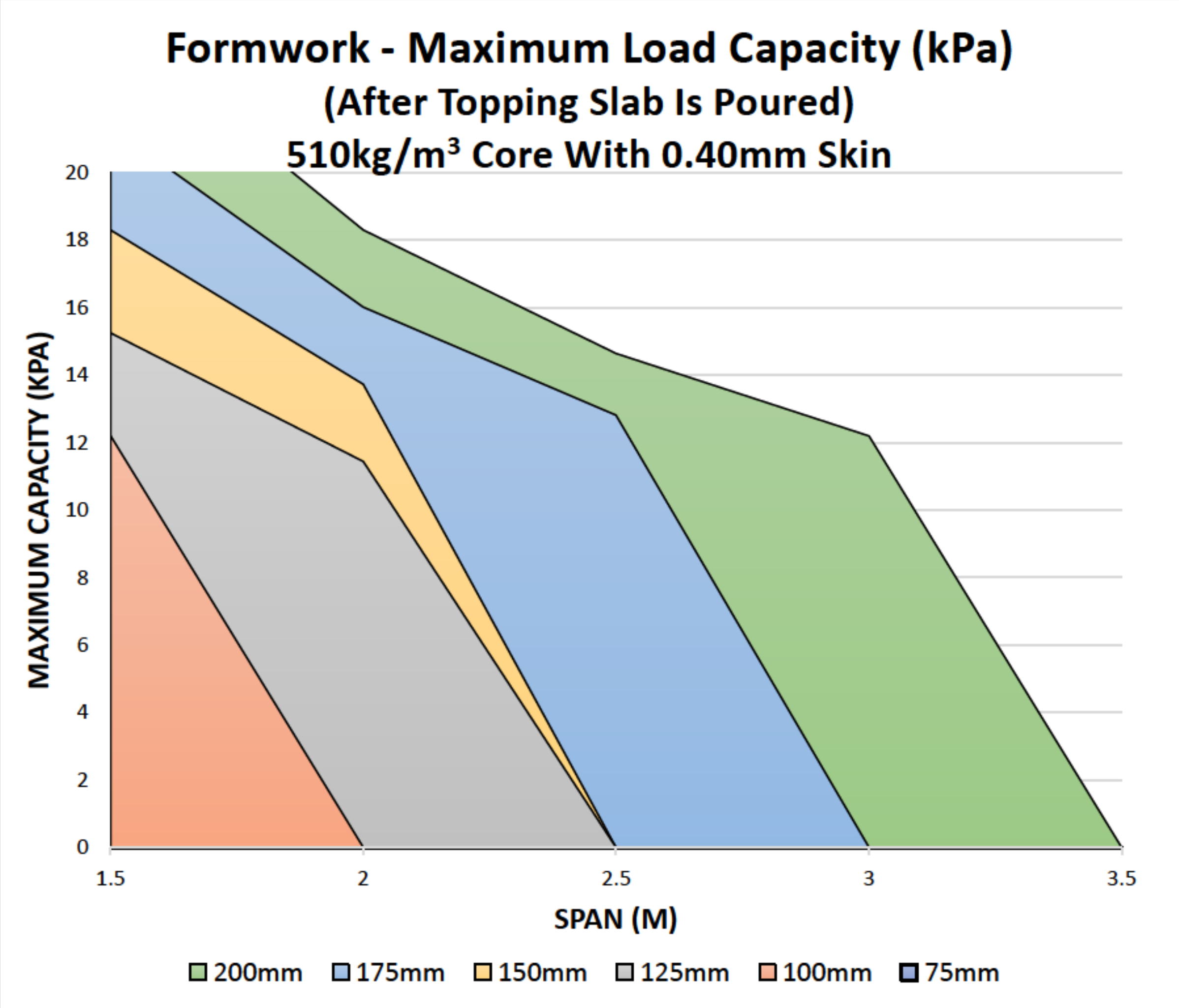


Figure N.5 - Load Capacity Graph - Formwork - 510kg/m3 - 0.40mm

Table N.6 - Span Table - Formwork - 510kg/m3 - 0.55mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.55 mm										
Panel Width 600 mm										
Core Density 510 kg/m³										
Core Thickness	**Span Length (m)**									
	1.5	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**	**6**
75mm	C	C	C	C	C	CF	CF	CF	CF	CFD
100mm	12.2	C	C	C	C	C	CF	CF	CF	CF
125mm	15.2	11.4	C	C	C	C	C	C	CF	CF
150mm	18.3	13.7	C	C	C	C	C	C	C	C
175mm	21.3	16.0	12.8	C	C	C	C	C	C	C
200mm	24.4	18.3	14.6	12.2	C	C	C	C	C	C

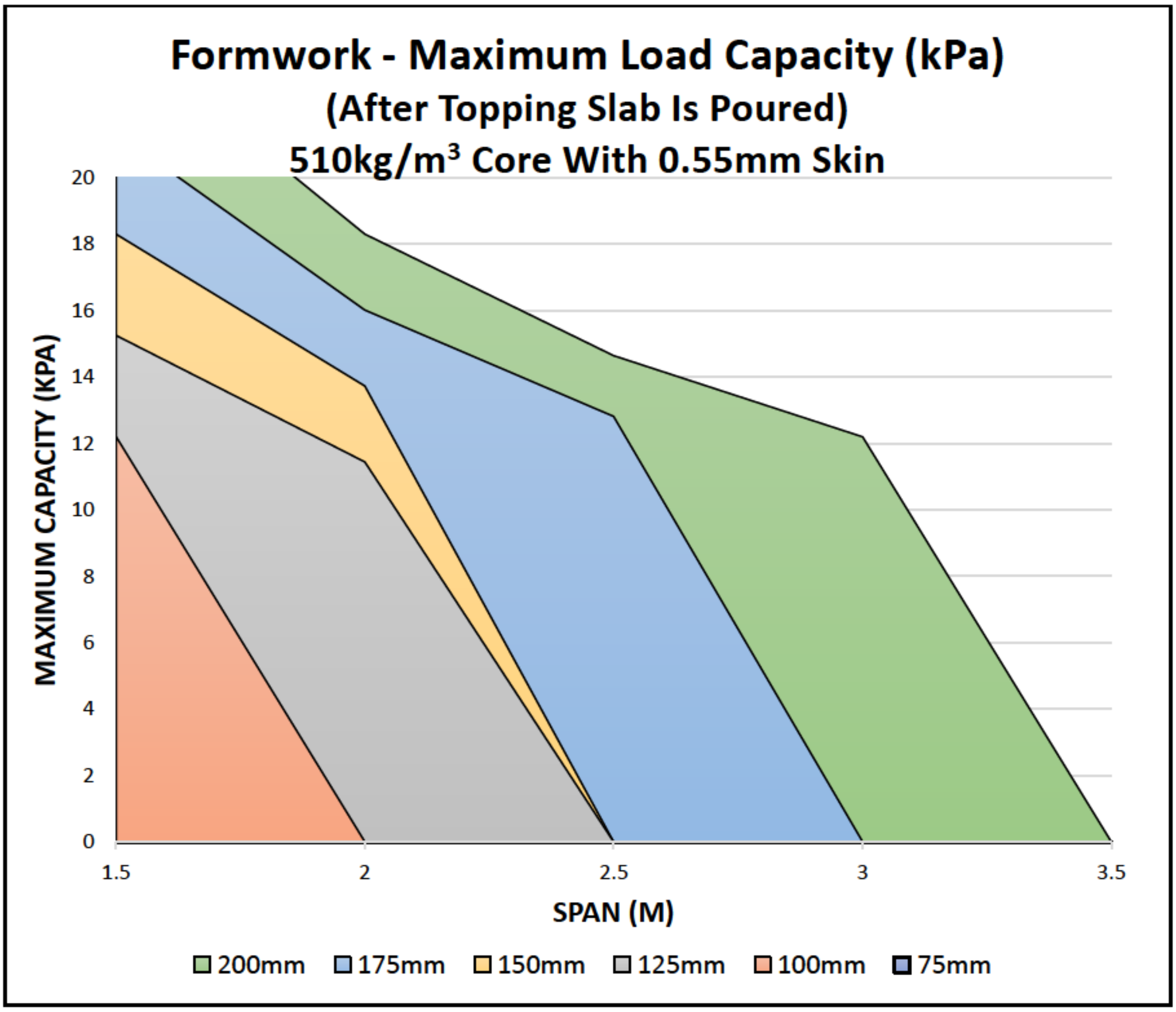


Figure N.6 - Load Capacity Graph - Formwork - 510kg/m3 - 0.55mm

Table N.7 - Span Table - Formwork - 510kg/m3 - 0.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.95 mm										
Panel Width 600 mm										
Core Density 510 kg/m³										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	C	C	C	C	C	C	CF	CF	CF	CF
100mm	12.2	C	C	C	C	C	C	C	CF	CF
125mm	15.2	11.4	C	C	C	C	C	C	C	CF
150mm	18.3	13.7	C	C	C	C	C	C	C	C
175mm	21.3	16.0	12.8	C	C	C	C	C	C	C
200mm	24.4	18.3	14.6	12.2	C	C	C	C	C	C

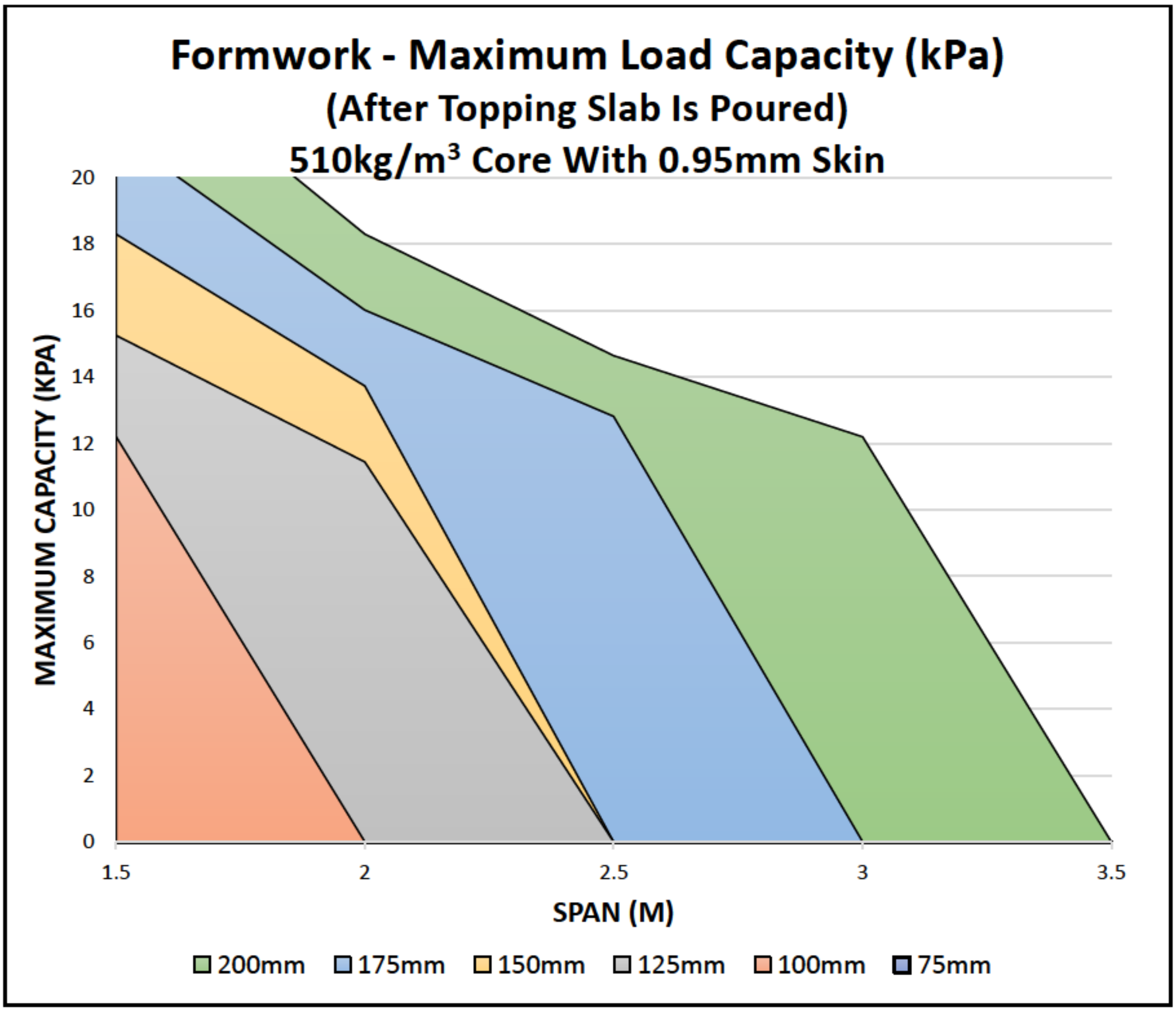


Figure N.7 - Load Capacity Graph - Formwork - 510kg/m3 - 0.95mm

Table N.8 - Span Table - Formwork - 510kg/m3 - 1.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 1.95 mm										
Panel Width 600 mm										
Core Density 510 kg/m^3										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	C	C	C	C	C	C	C	C	CF	CF
100mm	12.2	C	C	C	C	C	C	C	C	C
125mm	15.2	11.4	C	C	C	C	C	C	C	C
150mm	18.3	13.7	C	C	C	C	C	C	C	C
175mm	21.3	16.0	12.8	C	C	C	C	C	C	C
200mm	24.4	18.3	14.6	12.2	C	C	C	C	C	C

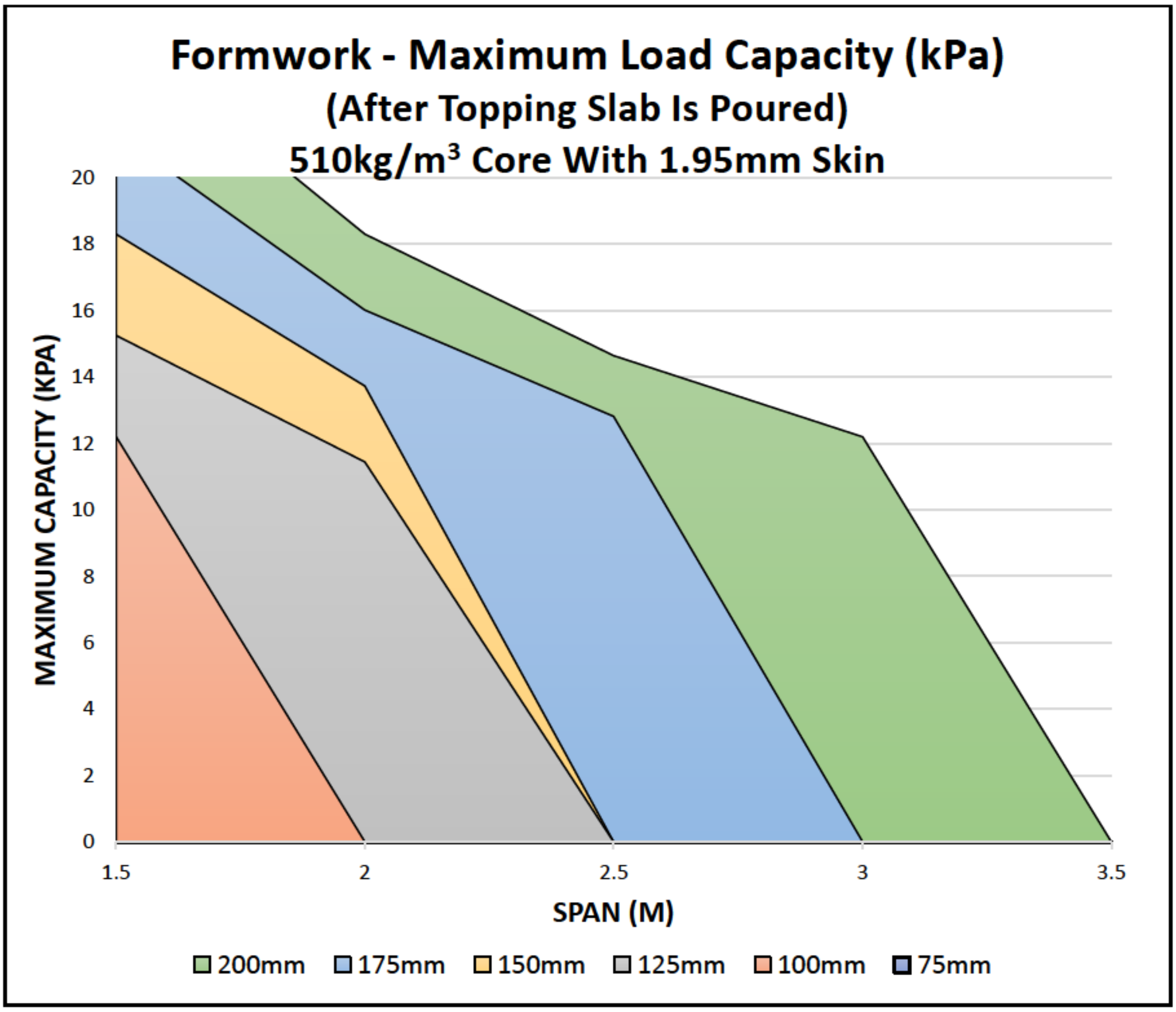


Figure N.8 - Load Capacity Graph - Formwork - 510kg/m3 - 1.95mm

Table N.9 - Span Table - Formwork - 550kg/m3 - 0.40mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.4 mm										
Panel Width 600 mm										
Core Density 550 kg/m³										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	CF	CF	CF	CF	CFD	CFD
100mm	17.4	13.1	C	C	C	C	CF	CF	CF	CF
125mm	21.8	16.3	13.1	C	C	C	C	CF	CF	CF
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	CF
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

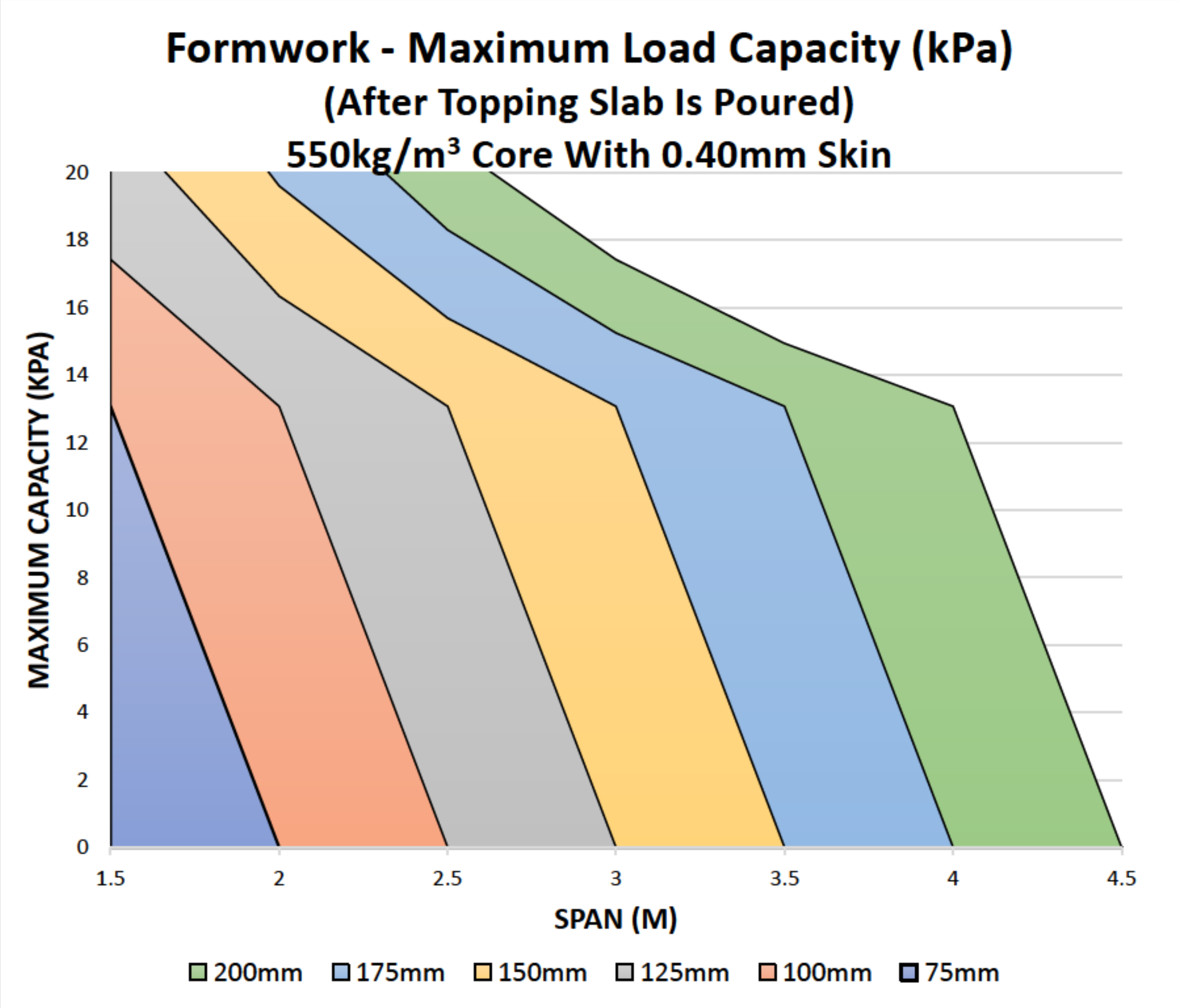


Figure N.9 - Load Capacity Graph - Formwork - 550kg/m3 - 0.40mm

Table N.10 - Span Table - Formwork - 550kg/m3 - 0.55mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.55 mm										
Panel Width 600 mm										
Core Density 550 kg/m^3										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	C	CF	CF	CF	CF	CFD
100mm	17.4	13.1	C	C	C	C	CF	CF	CF	CF
125mm	21.8	16.3	13.1	C	C	C	C	CF	CF	CF
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	C
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

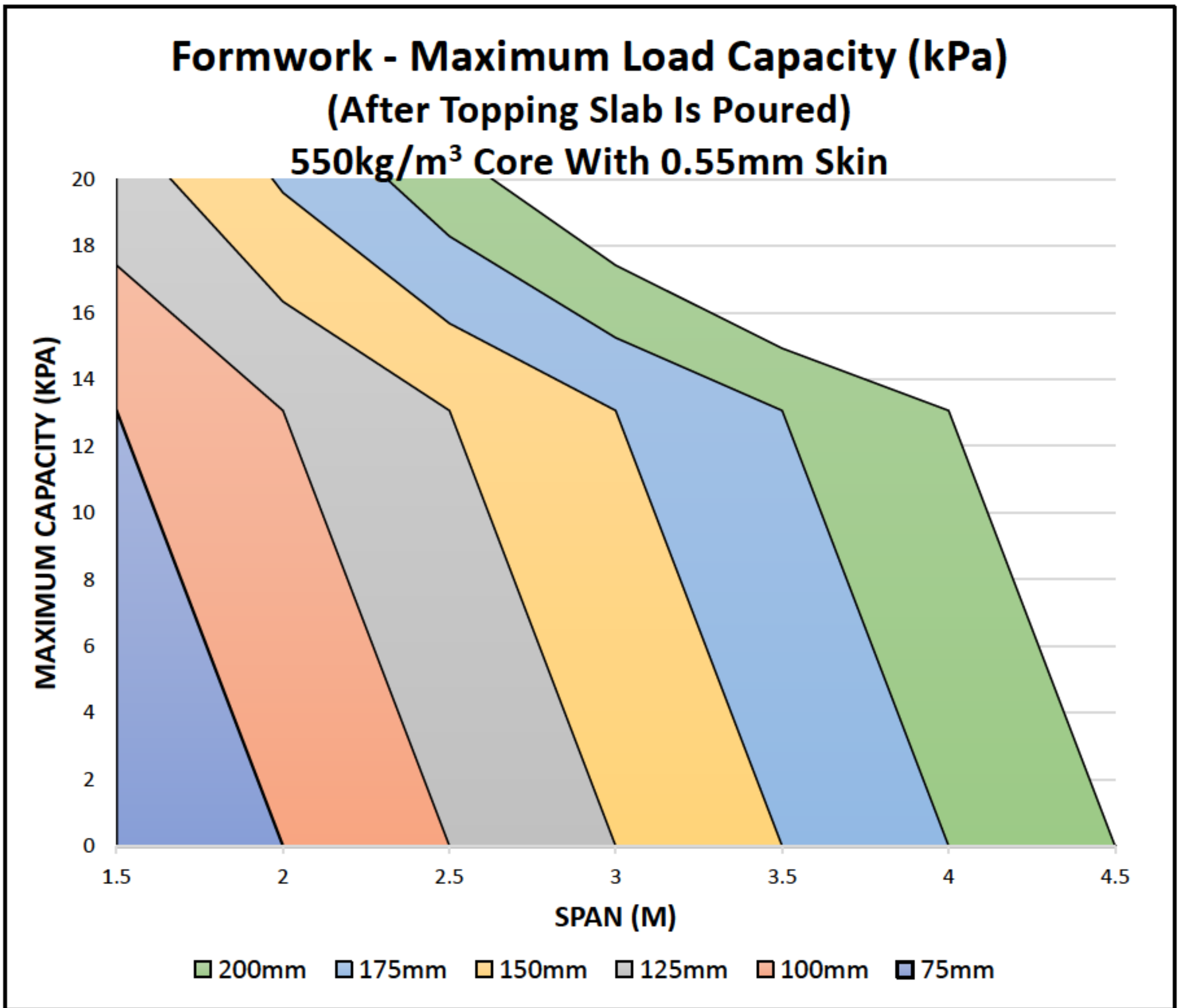


Figure N.10 - Load Capacity Graph - Formwork - 550kg/m3 - 0.55mm

Table N.11 - Span Table - Formwork - 550kg/m3 - 0.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.95	mm								
Panel Width	600	mm								
Core Density	550	kg/m^3								
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	C	C	CF	CF	CF	CF
100mm	17.4	13.1	C	C	C	C	C	C	CF	CF
125mm	21.8	16.3	13.1	C	C	C	C	C	C	CF
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	C
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

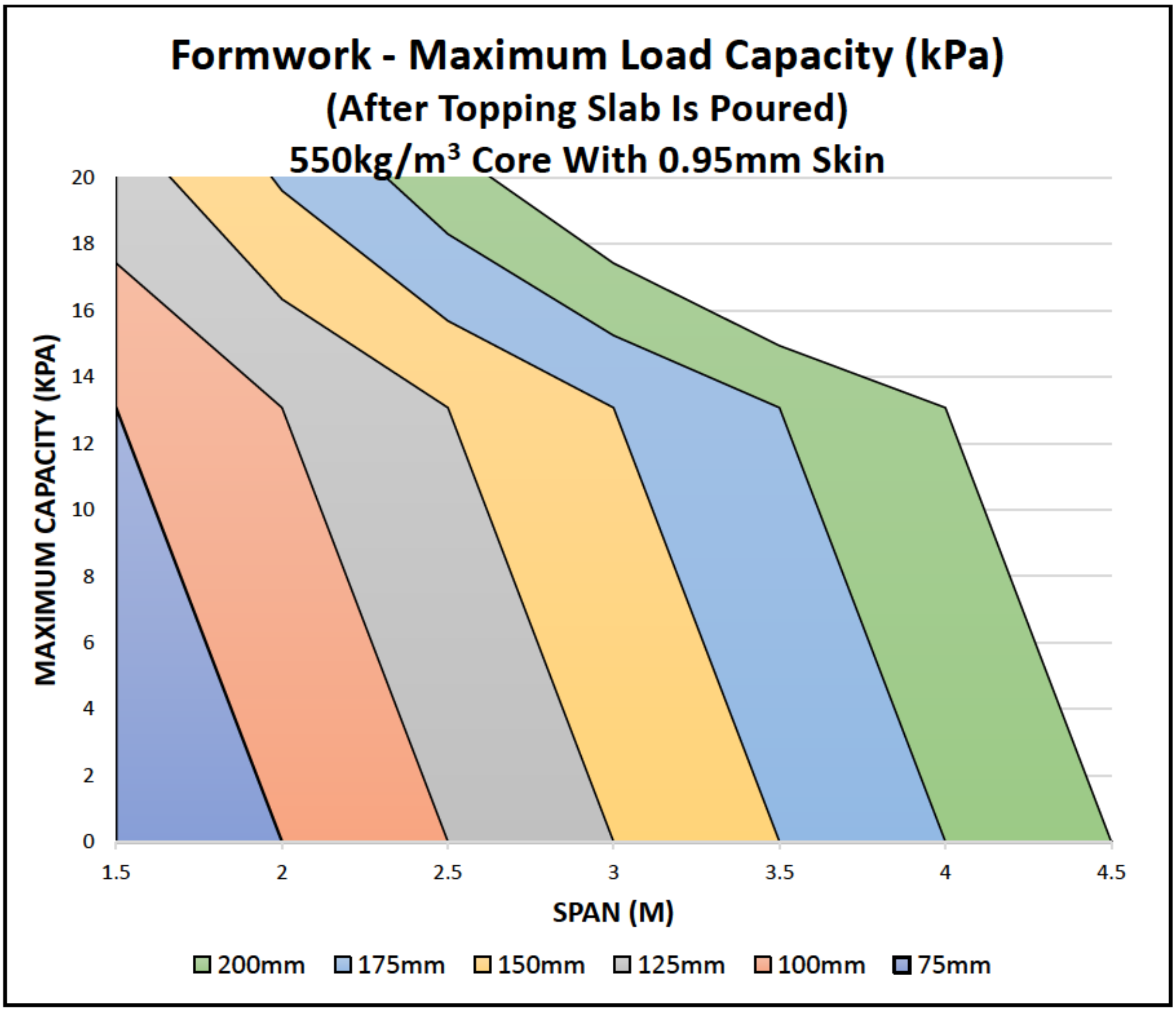


Figure N.11 - Load Capacity Graph - Formwork - 550kg/m3 - 0.95mm

Table N.12 - Span Table - Formwork - 550kg/m3 - 1.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	**1.95**	**mm**								
Panel Width	**600**	**mm**								
Core Density	**550**	**kg/m³**								
Core Thickness	**Span Length (m)**									
	1.5	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**	**5**	**5.5**	**6**
75mm	13.1	C	C	C	C	C	C	C	CF	CF
100mm	17.4	13.1	C	C	C	C	C	C	C	C
125mm	21.8	16.3	13.1	C	C	C	C	C	C	C
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	C
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

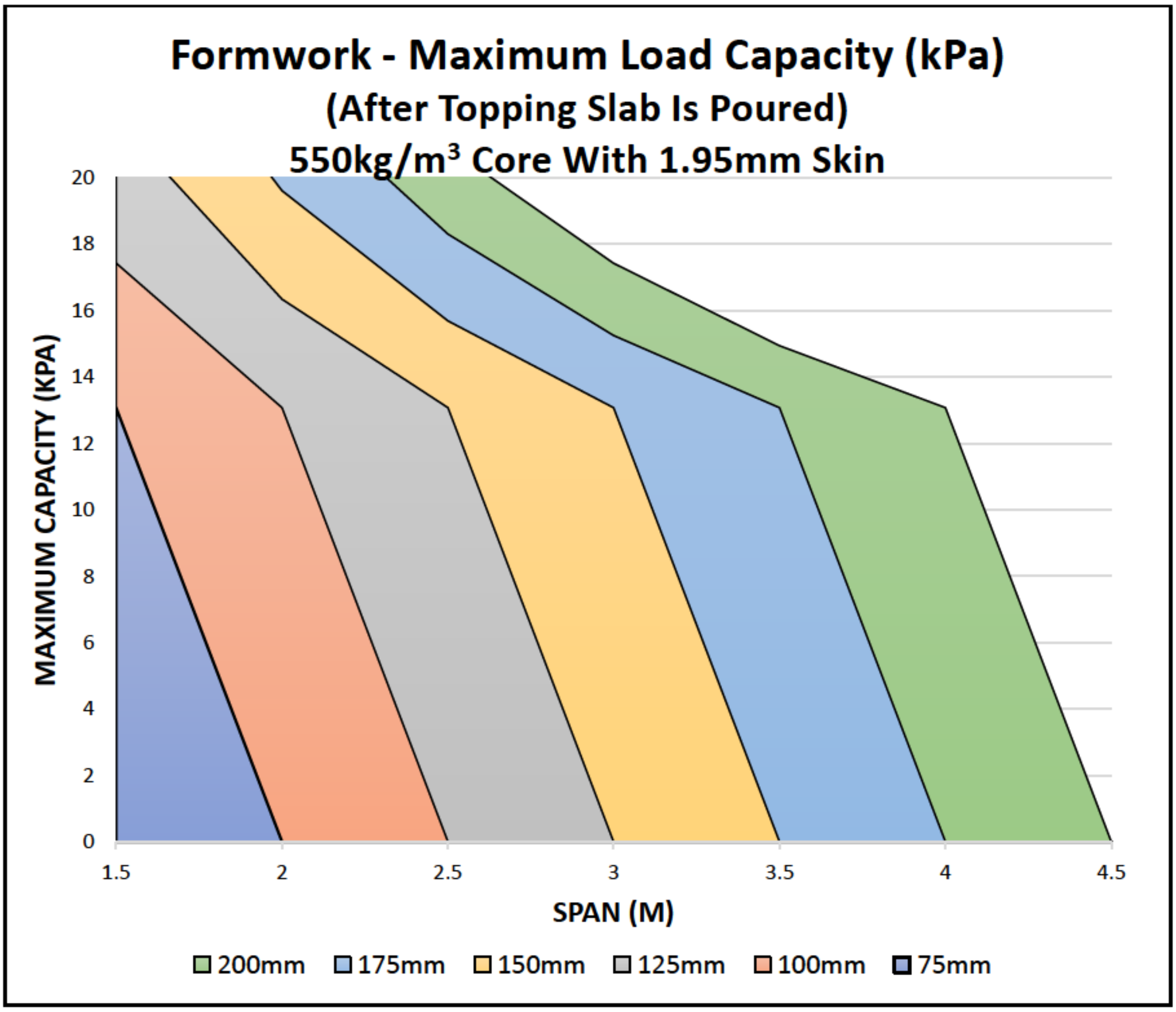


Figure N.12 - Load Capacity Graph - Formwork - 550kg/m3 - 1.95mm

Table N.13 - Span Table - Formwork - 580kg/m3 - 0.40mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.4 mm										
Panel Width 600 mm										
Core Density 580 kg/m^3										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	CF	CF	CF	CF	CFD	CFD
100mm	17.4	13.1	C	C	C	C	CF	CF	CF	CF
125mm	21.8	16.3	13.1	C	C	C	C	CF	CF	CF
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	CF
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

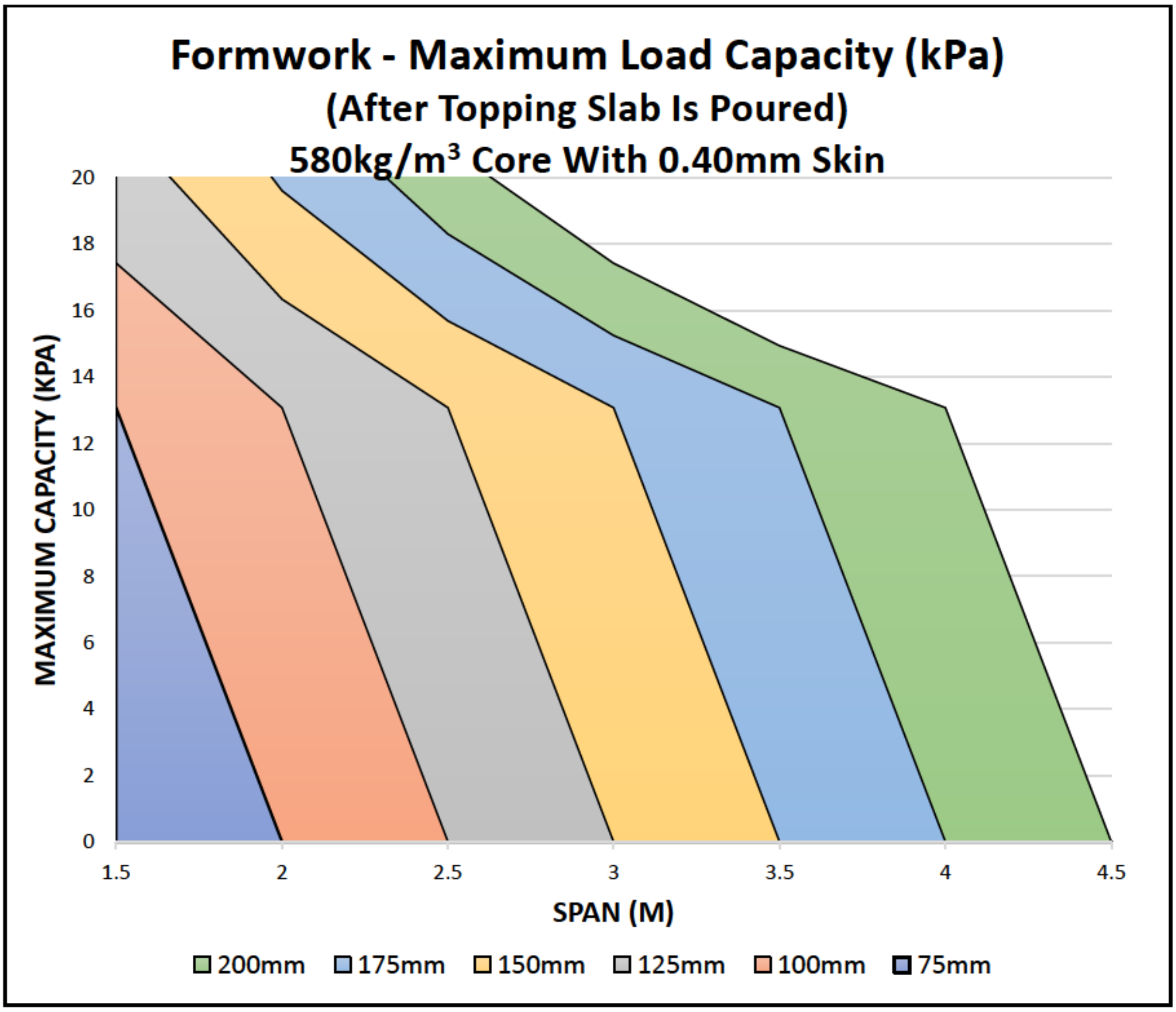


Figure N.13 - Load Capacity Graph - Formwork - 580kg/m3 - 0.40mm

Table N.14 - Span Table - Formwork - 580kg/m3 - 0.55mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness	0.55	mm								
Panel Width	600	mm								
Core Density	580	kg/m^3								
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	C	CF	CF	CF	CF	CFD
100mm	17.4	13.1	C	C	C	C	CF	CF	CF	CF
125mm	21.8	16.3	13.1	C	C	C	C	C	CF	CF
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	C
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

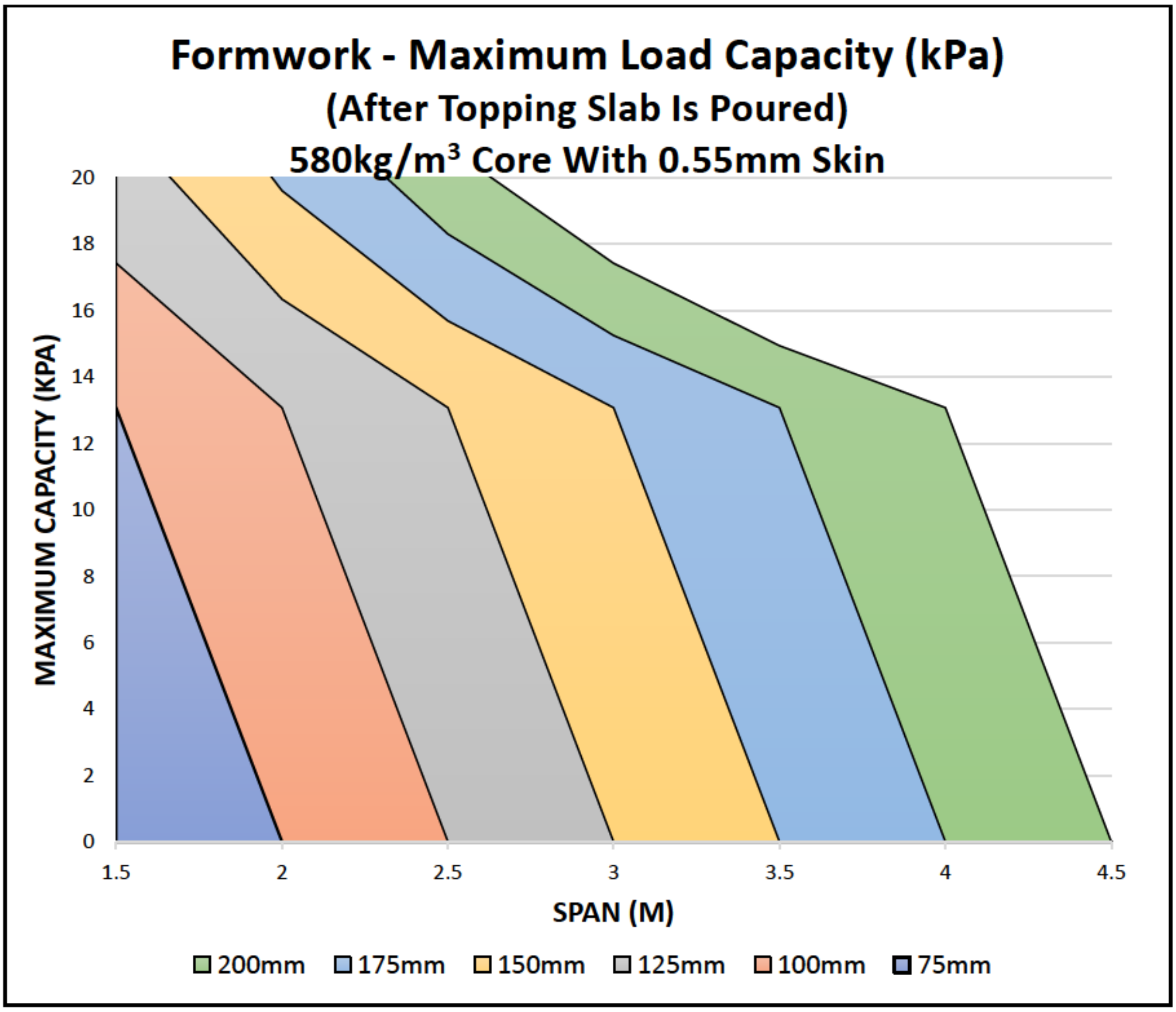


Figure N.14 - Load Capacity Graph - Formwork - 580kg/m3 - 0.55mm

Table N.15 - Span Table - Formwork - 580kg/m3 - 0.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 0.95 mm										
Panel Width 600 mm										
Core Density 580 kg/m^3										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	C	C	CF	CF	CF	CF
100mm	17.4	13.1	C	C	C	C	C	C	CF	CF
125mm	21.8	16.3	13.1	C	C	C	C	C	C	CF
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	C
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

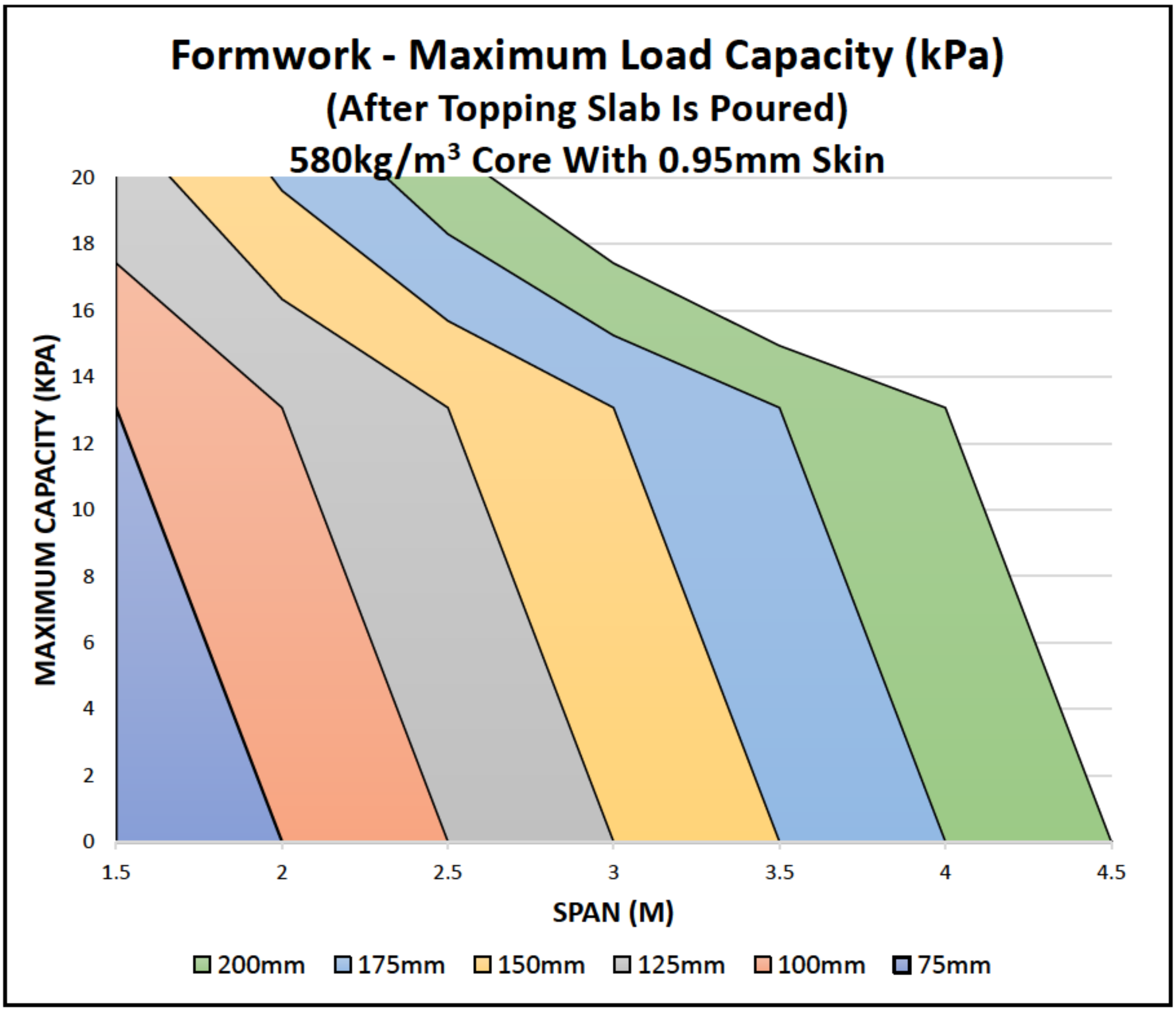


Figure N.15 - Load Capacity Graph - Formwork - 580kg/m3 - 0.95mm

Table N.16 - Span Table - Formwork - 580kg/m3 - 1.95mm

Formwork Application Maximum Floor Load Capacity (kPa) Table (After Topping Slab Is Poured) Single Span – Limited by Serviceability & Ultimate Limit State										
Sheet Thickness 1.95 mm										
Panel Width 600 mm										
Core Density 580 kg/m^3										
Core Thickness	Span Length (m)									
	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
75mm	13.1	C	C	C	C	C	C	C	CF	CF
100mm	17.4	13.1	C	C	C	C	C	C	C	C
125mm	21.8	16.3	13.1	C	C	C	C	C	C	C
150mm	26.1	19.6	15.7	13.1	C	C	C	C	C	C
175mm	30.5	22.9	18.3	15.2	13.1	C	C	C	C	C
200mm	34.8	26.1	20.9	17.4	14.9	13.1	C	C	C	C

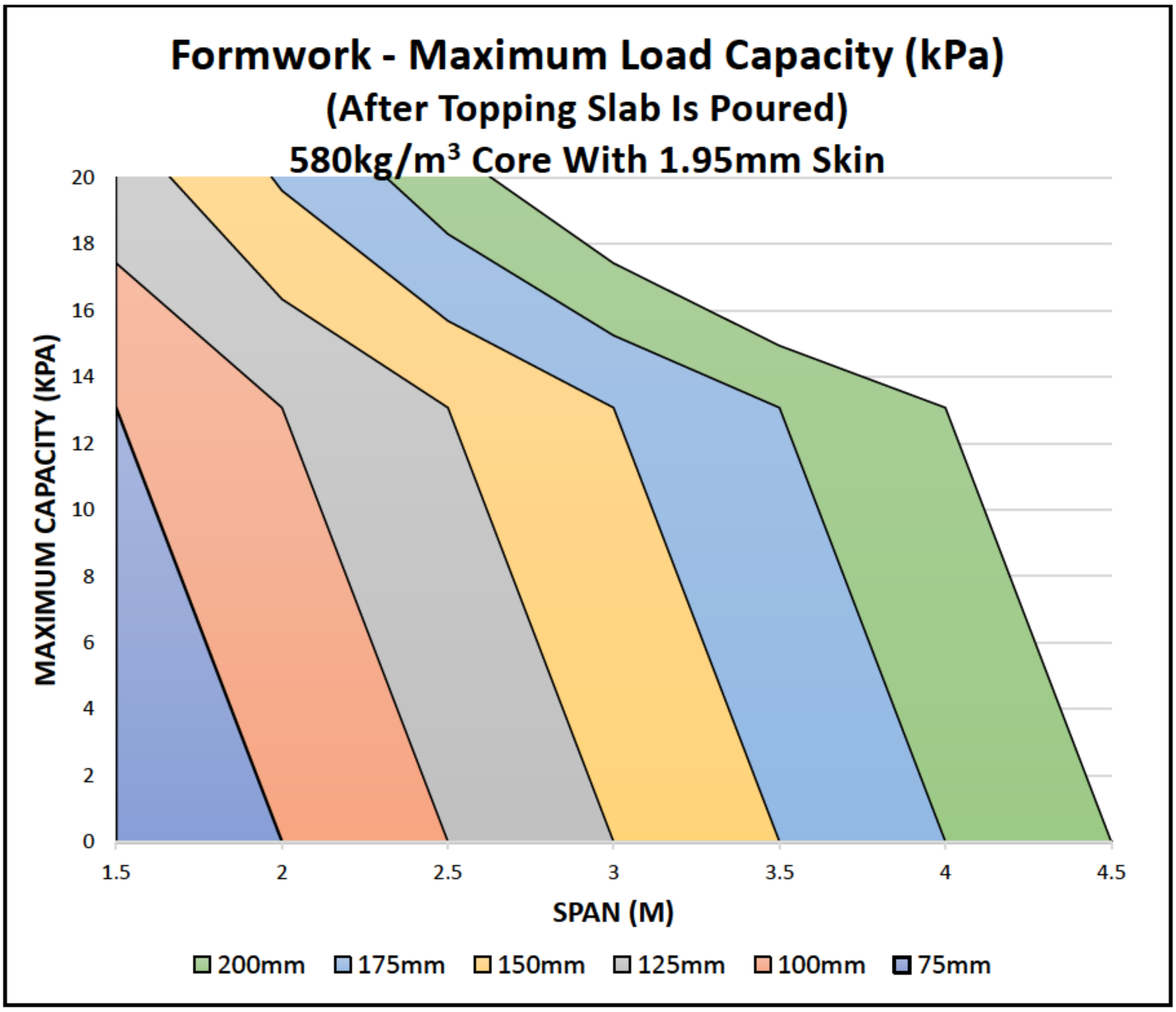


Figure N.16 - Load Capacity Graph - Formwork - 580kg/m3 - 1.95mm