Application of Sandwich Panels in Offshore Structures

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Huisman

Application of Sandwich Panels in Offshore Structures

Static Strength, Buckling Strength and Weight & Cost Analysis

by

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Abstract

In steel structures, a lot of attention is paid to lightweight structure, i.e. reduction of dead load without compromising structural safety, integrity and performance as well as cost-effectiveness. Thanks to modern steel aluminium sandwich panel manufacturing technology a new possibility became available for lightweight structural design.

The objective of this thesis is to evaluate the application of sandwich panels in the construction of steel structures with the aim of weight reduction without affecting other parameters like safety, performance, cost, etc. In this thesis, both column and plate buckling theories are considered and applied to the sandwich panel to evaluate its behaviour under in-plane compressive load. Effects of various material models and imperfections on buckling strength of sandwich panel are evaluated. Stiffened plate and sandwich panel is compared in terms of buckling resistance and self-weight. Three different sandwich panels made from faceplates of steel grade, S355, S690 & S1100, are used for replacement of S355 stiffened plate. Efforts made to understand the effect of various physical parameters on buckling resistance of sandwich panel in both column and plate buckling theories.

Finally, as a case study, sandwich panel technology is used to redesign the Huisman structure. The objective is to investigate whether applying sandwich panels in redesign makes it possible to obtain a sufficient weight reduction without losing its performance. For this case study, sandwich panels with faceplates of steel grade S355, S690 and S1100 are used. Static and buckling strength of the new design is evaluated. Also, the cost of new design and original design is evaluated and results are compared. Cost analysis is done to evaluate whether a sandwich panel is an economical solution.

Findings of this thesis are that in future it is possible to use sandwich panels in offshore structures to save a significant amount of weight while taking considerations into account. Sandwich panels can be successfully used to replace stiffened plates. Sandwich panels with faceplates made from extra high strength steel can give significant weight reduction. But the use of sandwich panels also results in an increase in the overall cost of the structure. So in terms of costs, it is questioned whether or not using sandwich panels is economically beneficial for offshore equipment.

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I would like to dedicate this report in the memory of my grandfather, Mr. Jayant Vidwans.

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List of Symbols & Abbreviations

Chapter 3

E_f	Modulus of Elasticity of Faceplates	$[MPa]$
I	Moment of Inertia of Sandwich Panel	$[mm^4]$
L	Buckling Length of Sandwich Panel	$[mm]$
λ	Slenderness of Sandwich Panel	$[-]$
A	Cross Section Area of Sandwich Panel excluding Area of Core	$[mm^2]$
N_{cr}	Critical Buckling Load	$[N]$
f_y	Yield Strength of Faceplates	$[MPa]$
φ	Value to determine the reduction factor χ	$[-]$
χ	Reduction factor for Column Buckling	$[-]$
$N_{b,Rd}$	Buckling Resistance of Sandwich Panel	$[N]$
N_{Ed}	Applied Load on Sandwich Panel	$[N]$
σ_E	Euler Buckling Stress	$[MPa]$
t_c	Thickness of Core of Sandwich Panel	$[mm]$
t_f	Thickness of Faceplate of Sandwich Panel	$[mm]$
t	Total thickness of Sandwich Panel	$[mm]$
b	Width of Sandwich Panel	$[mm]$
α	Imperfection Factor	$[-]$
$F_{buck,global}$	Buckling Load of Sandwich with Global Geometric Imperfection	$[N]$
$F_{buck,local}$	Buckling Load of Sandwich with Local Geometric Imperfection	$[N]$
F	Buckling Load of Sandwich with combined Local & Global Imperfections	$[N]$

Chapter 4.2.1

a	Length of Stiffened Plate (Base Plate)	$[mm]$
b	Width of Stiffened Plate (Base Plate)	$[mm]$
t	Thickness of Base Plate	$[mm]$
λ_p	Slenderness of Plate	$[-]$
$\sigma_{cr,p}$	Elastic Critical Buckling Stress of a Stiffened Plate	$[MPa]$
$\beta_{A,C}$	Ratio of Effective Cross-section Area to Gross Cross-section Area	$[-]$
f_y	Yield Strength of Stiffened Plate	$[MPa]$
$A_{c,eff,loc}$	Sum of Effective Areas of Sub-Panels and Stiffeners	$[mm^2]$
A_c	Gross Cross Section of Compression Zone of Stiffened Plate Excluding Edge parts along Longitudinal Edges	$[mm^2]$
$A_{sl,eff}$	Sum of Effective Areas of Longitudinal Stiffeners	$[mm^2]$
$b_{loc,i}$	Width of each individual sub-panel i	$[mm]$
$\rho_{loc,i}$	Reduction Factor of each sub-panel i	$[-]$
$k_{\sigma,p}$	Plate Buckling Coefficient	$[-]$
υ	Poisson's Ratio of Stiffened Plate	$[-]$

I_{sl}	Second Moment of Area of Whole Stiffened Plate	$[mm^4]$
I_p	Second Moment of Area of Base Plate	$[mm^4]$
A_{sl}	Sum of Gross Area of Individual Longitudinal Stiffeners	$[mm^2]$
A_p	Gross Cross Section Area of Base Plate	$[mm^2]$
σ_1	Larger Edge Stress	$[MPa]$
σ_2	Smaller Edge Stress	$[MPa]$
ρ	Reduction Factor	$[-]$
h_s	Height/Depth of Stiffener	$[mm]$
t_s	Thickness of Stiffener	$[mm]$
n	Number of Stiffeners	$[-]$
E	Modulus of Elasticity of Plate	$[MPa]$

Chapter 4.2.2

a	Length of Sandwich Panel	$[mm]$
b	Width of Sandwich Panel	$[mm]$
t_f	Thickness of Faceplate	$[mm]$
t_c	Thickness of Core	$[mm]$
N_{cr}	Critical Buckling Load per unit Width for Sandwich Panel	$[N/mm]$
$D_{1,2,3}$	Flexural Stiffness of Sandwich Panel	$[N-mm]$
m	Number of Half Sines in Longitudinal Direction	$[-]$
n	Number of Half Sines in Transverse Direction	$[-]$
υ_f	Poisson's Ratio for Faceplate	$[-]$
E_f	Modulus of Elasticity of Faceplates	$[MPa]$
G_c	Modulus of Rigidity of Core	$[MPa]$
E_{fx}, E_{fy}	Modulus of Elasticity of Faceplate in x and y direction respectively	$[MPa]$
ρ_f, ρ_c	Density of Faceplate and Core	$[g/mm^3]$
$\upsilon_{xy}, \upsilon_{yx}$	Poisson's Ratio	$[-]$

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1 Introduction

1.1 Sandwich Panels

Nowadays architects often need from structural engineers to design more slender steel structures, which is a challenging task. One of the reasons behind the slender structure is a reduction in dead load by design. Therefore, quite often common or most economic structure is not chosen, which results in challenges for structural engineers. In order to design an uncommon structure, it is important to have a proper understanding of these structures and their structural behaviour

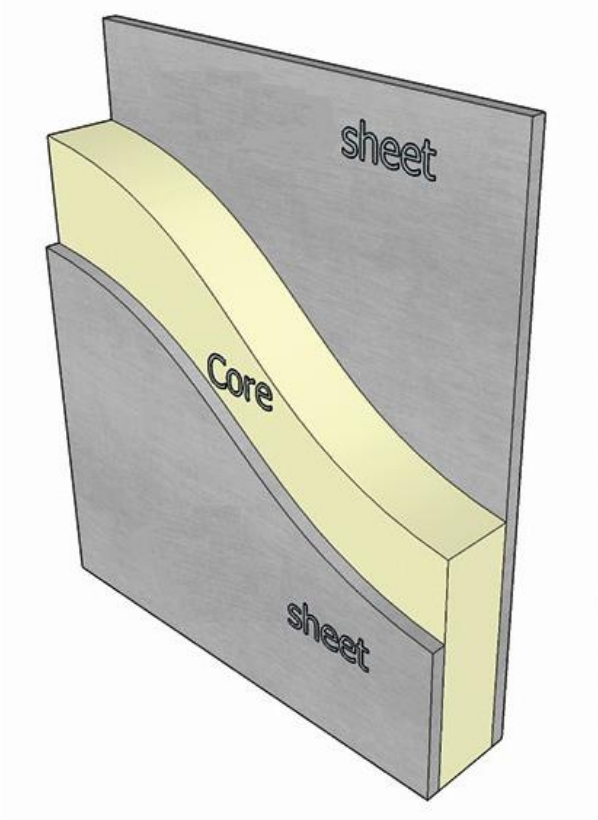


Figure 1.1 Sandwich Panel

The thesis deals with the study of the application of steel aluminium foam sandwich panel in steel structure. Main advantages of sandwich construction, development of new material and need for low weight, high-performance structures insure that sandwich construction will continue to be in demand. Sandwich panels are mainly used in the ship industry or in the automobile industry. Use of sandwich structures continues to increase rapidly for application ranging from automobile, ship, aircraft, satellites, wind energy system, rails carts and bridge construction to mention only a few. There is no significant application of sandwich panel in the construction industry as a major structural component/member. Therefore, the industry lacks documents and references for the sandwich panel.

1.2 Problem Statement

Huisman Equipment B.V. is a specialist in the field of heavy transport, heavy lifting systems, special handling systems and offshore equipment. One of their products is heavy-lift equipment, which can be installed on different types of vessels and is capable of lifting heavy loads. To obtain and maintain competitive advantage the company is always looking for innovative and creative ways to satisfy the needs of their clients, to lift more and higher while using less material. In offshore industry, weight reduction is one of the key elements in designing of lifting equipment. At Huisman for weight reduction, stiffened plated structures and extra high strength steel S690 is used quite often.

Aim of weight reduction has strived need for the development of new material. This lead to the application of sandwich panels in steel structures. Same as stiffened plates, significant weight reduction can also be achieved by the use of sandwich panels. Use of sandwich panels can result in a lot of advantages such as,

- Increased payload of a floating vessel
- Increased lifting capacity
- Allow further outreach
- More flexibility of crane reduces dynamic impacts
- Less steel usage may imply fewer costs on material and fabrication
- Less strong erection equipment required
- Ease in installation

Objectives behind this master thesis are,

1. Understand the behaviour of the sandwich panel when subjected to in-plane compressive load.
2. Find out the effect of various parameters on the buckling of sandwich panel.
3. Comparison of sandwich panel with a stiffened plate in terms of bucking, weight, cost.
4. Finding out the effect and relation of various properties of a sandwich panel to obtain optimal design.
5. Case study of Huisman structure. Redesigning structure with help of sandwich panels. Performing static and buckling check. Evaluating if weight reduction is achieved or not. Evaluating the cost of structure designed with sandwich panels. Comparing the cost of a new design with an original design.

1.3 Reader's Guide

Chapter 1 gives a general introduction and outline of the thesis. Also, a short description of a problem tackled in the thesis.

In chapter 2 overview of the current state of the art in a sandwich panel is presented.

Chapter 3 deals with the application of column buckling to the sandwich panel. Various parameter such as material models and geometric imperfections, which affect the behaviour of sandwich, are explained. In addition, the effects of these parameters on behaviour are explained. Outcomes of analysis are compared.

Chapter 4 deals with an application of plate buckling to the sandwich panel. The resistance of the sandwich panel and stiffened plate are calculated and compared. The sandwich panel and stiffened plate are compared in terms of self-weight. Also, attempts made to replace stiffened plate by different sandwich panels with different steel grades.

Chapter 5 explains various ways to optimise the design of the sandwich panel. Effect of various parameters such as the thickness of faceplates, the thickness of core and yield strength of faceplates on resistance and weight of the sandwich panel is explained.

In Chapter 6 representative case study example of Huisman structure is introduced as a design example considered for application of sandwich panel. The original structure is redesigned with the help of sandwich panels. Static check, buckling check and cost analysis is performed. Outcomes are compared with the original design.

In Chapter 7 conclusion of this study and recommendations for further research will be presented

2 Sandwich Panels - State of Art

The most simple type of sandwich panel consists of two strong, stiff, thin plates/sheets of highly dense material separated by a thick layer of low-density material that can be much less strong and stiff[2]. Following Figure 2.1 will give an idea of a sandwich panel. Load-carrying faceplates are separated by a core with low density, which results in an increase in moment of inertia of sandwich panel with a small increase in self-weight, making it an efficient structure. Most often sandwich panel has two identical faceplates, which are separated by a core. However, in certain special circumstances thickness of faceplates, the material used or both might differ. Sandwich with two identical faces is regarded as a mid-plane symmetric sandwich, while sandwich with different faces is a mid-plane asymmetric sandwich. [1]

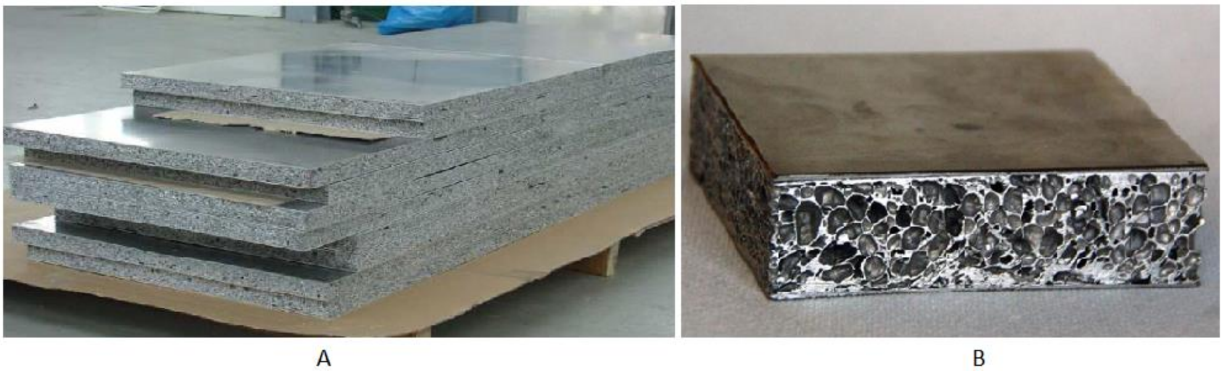


Figure 2.1 Steel Aluminium Foam Sandwich Panel [5][6]

Wide range of advantages justifies rising interest in the use of sandwich panels in the construction of various structural systems as well as buildings. Some advantages of sandwich panel are high bending stiffness, high load carrying capacity and high strength to weight ratios, reduction in the cost of formwork & foundation and high structural efficiency [20]. These panels are also efficient in thermal and sound insulation. By using these lightweight materials in the transport industry, the payload can be raised, higher speeds can be reached and less fuel consumption can be obtained [21]. Some of the other advantages of sandwich panel are mass predictability, fast erect-ability, long-spanning capability, durability, pre-fabricability and finally yet importantly reusability. These characteristics make the sandwich panel very useful in places with unfriendly environments where erection time and labour needs to be minimised [7].

2.1 History

Bert, Noor and Burton state that concept behind sandwich construction can be traced back to Fairbairn in England in 1849[1]. The first application of sandwich technology was in Mosquito aircraft in England, which was used in the Second World War [2]. In mosquito aircraft, plywood sandwich construction is used.

In the United States, the concept of sandwich construction with a low-density core and faceplates made of reinforced plastic is originated. In 1943, Vultee BT-15 fuselage was designed and fabricated by Wright Patterson Air Force Base. It was made using both a glass fabric honeycomb and balsa core and using fibreglass-reinforced polyester as face material [1].

In the 1960s, sandwich technology is mostly applied in the aerospace industry. In 1969, the first successful landing of a space ship on the moon took place [4]. This was a result of the application of various technologies such as rocket, aerospace, computer science and last but not least sandwich construction. Sandwich construction made the landing of this space ship possible. This is due to the fact that due to sandwich structure, weight of space ship is reduced but at the same time was able to resist stresses or loads applied on a rocket, examples can be air/wind pressure, landing & take-off stress, etc.

In 1992, Bitzer of Hexcel gave an overview of a honeycomb core material and their application. Bitzer states that in the western world some honeycomb core sandwich is been utilised by every two-engine aircraft. In Boeing 707, only 8 % of the wetted surface is sandwich whereas in newer Boeing 757/767 46% of the wetted surface is honeycomb sandwich. Fuselage cylindrical shell of Boeing 747 is primarily made from honeycomb sandwich. Along with fuselage cylindrical shell, ceiling, side panels and floors of Boeing 747 are also made from sandwich construction [1].

Afterwards, because of such success, the number of other applications of sandwich panels were discovered in various fields such as building, ship, automobile industry, etc.

2.2 Type of Sandwich Panels

A lot of sandwich construction can be made based on structural requirements by combining different core and face material. Facing are usually made up of aluminium, steel, fibre reinforced polymer, wood or even concrete. The core can be made of solid plastic materials, polyurethane, balsa wood and metal foam such as aluminium.

This possibility of combining materials makes it possible to make an optimum structure of the sandwich panel for specific applications. In sandwich panels, it is possible to combine the positive properties of individual materials. This freedom makes it possible to make a sandwich panel with the following favourable properties.

- High load-bearing capacity with low self-weight
- Capacity for rapid erection without heavy lift cranes or equipment
- Ease in installation
- Easy to replacement or repair in case of damage
- Long-life at low maintenance cost

2.2.1 Faceplate Material

Thin sheets/plates with high strength are generally used as facing materials. These sheets must meet manufacturing requirements with regard to bending and roll forming, fictional requirement with regard to water, wind and vapour tightness, structural requirements in their capacity as a component of a composite panel and their ability to resist local loads and furthermore they must have adequate resistance to fire and corrosion. Not all of the requirements mentioned above are important in given application but it is clear that metal sheets particularly steel and aluminium economically satisfy them.

A thin plate of steel is most commonly used in facing material. In case of steel plates, coating of zinc-aluminium or aluminium-zinc alloy can be allied to steel sheeting as a metallic corrosion protection layer. At the same time, a perfect bond between face and core should be achieved.

In an application where the special requirement of corrosion resistance or hygiene is required, a sandwich panel with aluminium faceplates can be used. Modulus of elasticity of aluminium is one-third of steel and density is also one-third of that steel. On the other hand, the coefficient of thermal expansion is nearly twice that of steel.

In the case where an attack from the environment has to be resisted or hygienic demands are high, stainless steel faceplates may be used. Stainless steel or copper plates can be used where high quality and maintenance-free sandwich panel are required. In this case of stainless steel or copper, no corrosion protection is required.

So far these type of panels mainly has semi-structural character. In the building industry, panels have the function of carrying relatively small loads over fairly long spans. Building panels should be lightweight like aircraft panels, but unlike aircraft panels, they should be cheap. In the building industry, all-metal panels have a substantial application but panels with other materials also have the scope of application. For faces there are glass reinforced plastics, resin impregnated paper, glass-reinforced cement, asbestos cement, plywood, plasterboards, ferro cement and hardboard. Table 2.1 shows the properties of various face materials,

X = strength, T = tension, C = compression, ρ = density, E = Young's modulus, G = shear modulus, ν = Poisson's ratio.

Material	ρ g/cm^3	E GPa	G GPa	ν	X_T MPa	X_c MPa
Aluminum (2024-T3)	2.80	73	27.4	0.33	414	414
Steel (AISI 1025)	7.80	207	80.0	0.30	394	394
Titanium	4.40	108	42.4	0.30	550	475
S-Glass/EP[1]	1.73	20.6	3.10	0.12	261	177
E-Glass/EP[1]	2.00	26.6	4.63	0.144	422	410
AS4-Carbon/EP[1]	1.63	59.5	4.96	0.047	584	491

[1]The composites consist of woven 0 and 90°fibers in an epoxy (EP) matrix.

Table 2.1 Mechanical Properties of Face Materials [8]

2.2.2 Core Material

Core materials must have appropriate properties such as mechanical strength, stiffness so as to achieve a proper sandwich panel. The core is generally made up of inorganic fibre material, rigid plastic foam or metal foam. Relevant mechanical properties can be tensile, compressive, shear strength and modulus of elasticity. Ambient temperature and humidity have an effect on the properties of the polymeric core but not on metal foam core. Therefore, the choice of the core is affected by required core properties such as resistance to moisture, thermal & sound insulation and performance in fire.

The core has several vital functions. To ensure that faceplates remain at correct distance apart, the core should have proper stiffness in the direction perpendicular to faceplates. Shear stiffness of core plays an important role in bending. Core shear stiffness is important so as to avoid sliding of faces over each other during panel bending. If panel sliding it not restrained then, faceplates behave as two independent panels or beams. This results in loss of sandwich effect. Core stiffness also affects local buckling of faceplates. The core should be stiff enough to keep faceplates flat, if this is not fulfilled then it can result in local buckling of faceplates under application of in-plane compressive load. Therefore, it is important that the core satisfies all these requirements. Also, it is important that the flexibility of the bond between core and faces is considered. Bond should not be flexible enough to permit relative movement between core and faceplates.[2]

Core with appropriate stiffness can make a useful contribution in bending stiffness of sandwich as a whole.

The core must be stiff and strong under shear and extension in the direction of thickness to avoid failure due to wrinkling and due to local indentation as shown in Figure 2.2. At the same time, to reduce the weight of the structure, core should have low density. These types of demands are conflicting with each other since materials with low density are less strong and stiff than materials with higher density. Ashby's materials property charts (Ashby 1999) can be used as a guide for selections of core and face materials. Following Figure 2.3 shows an example of such chart [8].

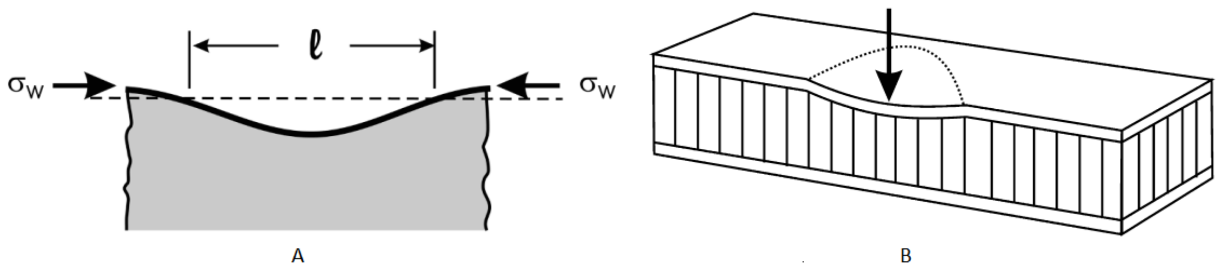


Figure 2.2 A) Wrinkling of face sheets loaded in compression B) Local indentation failure due to concentrated load [8]

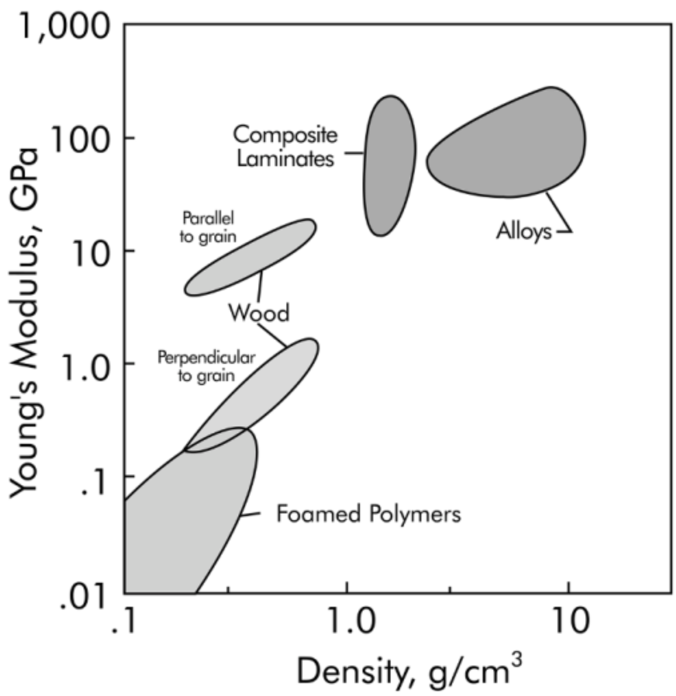


Figure 2.3 Modulus density chart for various classes of materials. After Ashby 1999 [8]

If cores have low density, its contribution can be considered small and it might be very convenient to ignore it. This also makes stress and deflection analysis considerably simple than a case where the contribution of the core is considered. As a preliminary guide to proportions, when the combined weight of faces roughly equals the weight of core an efficient sandwich panel is obtained. If bending stiffness of this arrangement is compared with single solid plate, then it is observed that solid plate has very low bending stiffness compared to a sandwich whereas it has the same weight as that of the sandwich panel [2].

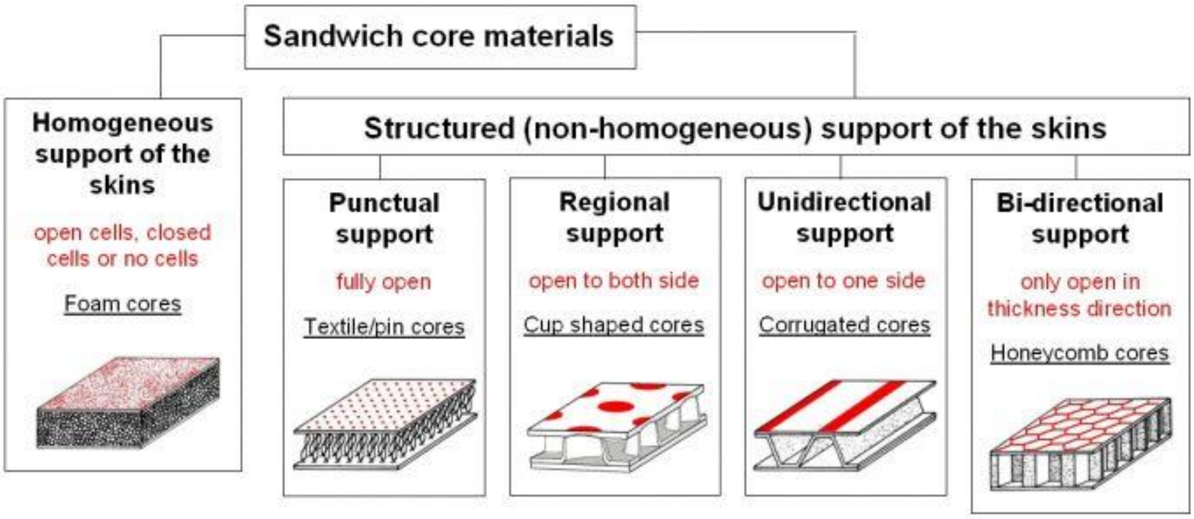


Figure 2.4 Types of material used for core [25]

Following Table 2.2, Table 2.3 & Table 2.4 shows the properties of various core materials,

S = shear strength, W = width direction, ρ = density, G = shear modulus, L = length direction. From Zenkert (1997).

Material	ρ g/cm^3	G_L MPa	G_W MPa	S_L MPa	S_W MPa
Paper	0.056	141	38	1.3	0.48
Aluminum	0.070	460	200	2.2	1.50
Nomex	0.080	69	44	2.2	1.00
Nomex	0.129	112	64	3.2	1.70

Table 2.2 Mechanical Properties of honeycomb core [8]

ρ = density, G = shear mod-ulus (out-of-plane), S = shear strength (out-of-plane).

Product Designation	ρ g/cm^3	G MPa	S MPa
SB50	0.100	110	1.91
SB100	0.151	157	2.94
SB150	0.244	302	4.85

Table 2.3 Mechanical Properties of balsa wood core [8]

ρ = density, G = shear modulus, S = shear strength. Data obtained from Zenkert (1997), DIAB and Rohacell.

Material	ρ g/cm^3	G MPa	S MPa
Polyurethane	0.04	4	0.25
PVC H100	0.10	40	1.40
PVC HD130	0.13	40	1.50
PMI 110IG	0.11	50	2.40

Table 2.4 Mechanical Properties of polymer core [8]

2.3 Previous Research

Considerable research has been carried out on sandwich panels. But the majority of this research is on the sandwich with faceplates made of FRP, glass fibre or metal plates and a core made from honeycomb, polymer or balsa wood. Though this thesis focuses on steel aluminium foam sandwich panel, still research on other types of sandwich panels can be considered for reference.

2.3.1 Local Buckling Strength of Steel Foam Sandwich Panels

This paper provides and verifies a new method of designing for in-plane compressive strength of steel sandwich panel comprised of the steel faceplate and steel foam core. Steel foam core provides enhanced bending rigidity, potential to mitigate local instability and exceptional energy dissipation. Winter’s effective width expression is generalised to the case of steel foam sandwich panel.
For verification, LS-DYNA brick model was stimulated. The result indicates that it is important to include shear effects in solution and if shear effects are included then Winter’s method can be used for accurate strength prediction.

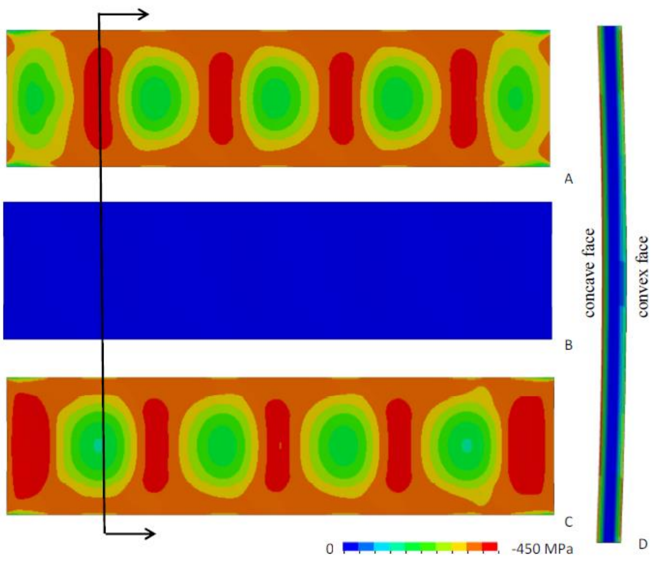


Figure 2.5 In-plane stress distribution in a panel: A. top face (steel plate), B. mid-plane (foam plate), C. top face (steel plate), D. cross-section (top steel - steel foam - bottom steel face) [5]

As per Figure 2.5, in faceplates stress varies along length, also increase and decrease in stress is observed as it follows buckling waves. At the centre of the foam core, stress is zero. This contradicts with stress variation in the faceplate. In faceplate, at centre high net compression is observed. If longitudinal stress is cut in the transverse direction this stress variation can be observed in more detail. This stress variation is shown in Figure 2.5 and in Figure 2.6. Stress distribution in Figure 2.6 can be readily recognised as similar to classic stress distribution, which motivated effective width expression of von Karman and later Winter.

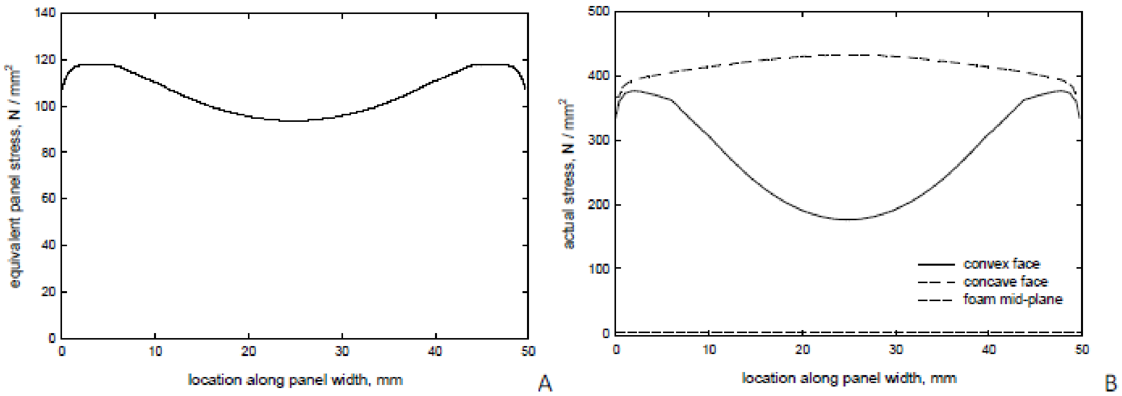


Figure 2.6 Resistance mechanism of sandwich panels: A. integral through-thickness (effective compressive resistance) expressed in terms of equivalent smeared stress, B. stress distribution in convex steel face, concave steel face and foam mid-plane.

Different conclusions of the paper are improved bending rigidity, high stiffness to weight ratio, etc. for steel foam sandwich panel consisting of steel foam in between two steel faceplate. Also, steel foam limits loss in yield stress and effective modulus. Effective width method proposed by Winter can be modified and applied to steel foam sandwich panel.[5]

2.3.2 Production Technology

There are several ways for manufacturing of sandwich panel, which are explained in short in chapter 2.5. Powder metallurgy process is one of many processes, which is used for manufacturing of sandwich panel. Powder metallurgy process is developed and implemented by Fraunhofer Institution for Manufacturing and Applied Material Research, Bremen [16], and is shown in the following Figure 2.7. Foaming agents and aluminium powder are mixed together using conventional mixers. Afterwards, this mixture is compacted into dense non-porous foamable solid aluminium.

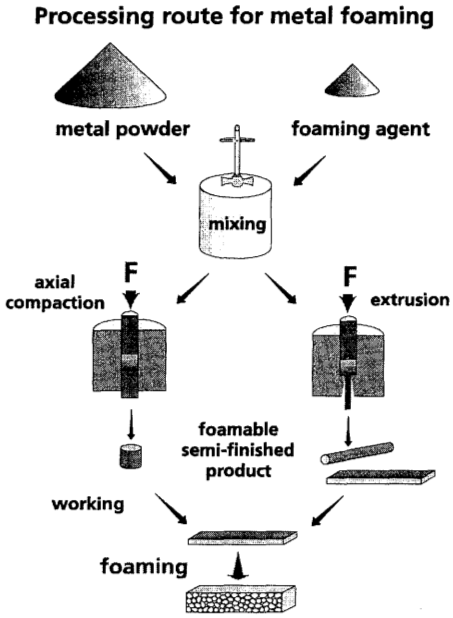


Figure 2.7 Production of metal foams (powder metallurgical IFAM-Process) [16]

Foamable aluminium is joined with aluminium plated steel faceplates by rolling as shown in the following Figure 2.8. Rolling can be done as warm or cold rolling with different equipment, with required deformation and after surface treatment [17]. Finally, foamable material is heated up to the melting point of faceplates which initiates its expansion into a sandwich panel of desired dimensions. The porosity of foam is high up to 80 to 90 % so very low density up to 0.7 gram/cm^3 can be achieved.

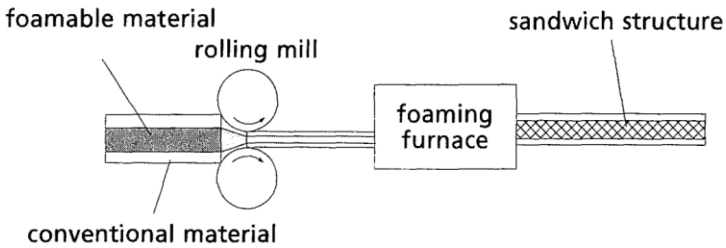


Figure 2.8 Schematic processes for of production of steel sandwich with aluminium foam core [18]

It is possible that foaming of sandwich panel can be done in continuous belt furnace which allows for the production of larger quantities of material. The main principle of continuous belt furnace is shown in the following Figure 2.9. In this process, there will be three heating zones, which can be operated and controlled separately/individually such that necessary temperature profiles for the foaming step can be obtained.

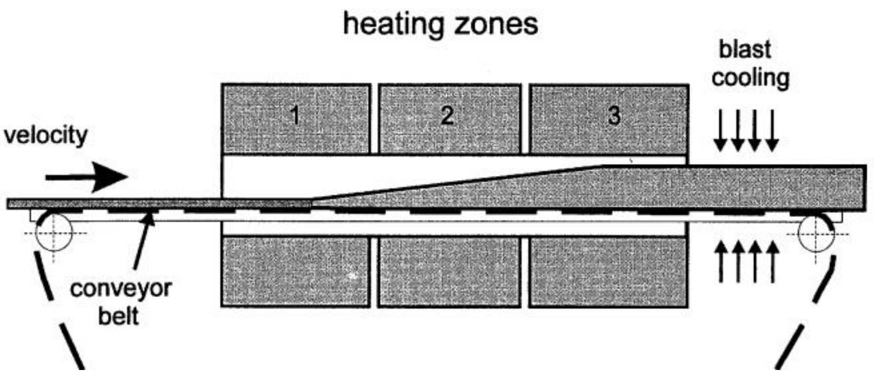


Figure 2.9 Schematic principle of continuous belt furnace for foaming of the core layer [36]

Using these methods continuous large-scale manufacturing of sandwich panel with aluminium foam produced by powder metallurgical foaming processes is possible [19].

2.4 Failure of Sandwich Panel

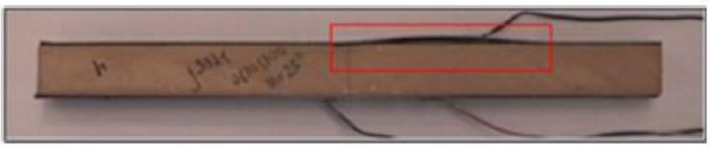
The sandwich panel is made from different material with different material properties, therefore it is hard to predict structural behaviour and properties of the sandwich panel. This is the reason why the sandwich panel shows different failure modes under different types of loading or a combination of different failure modes for a combination of different types of loading. Failure of the sandwich panel is influenced by types of loading, geometric dimensions and properties of components. Because of nonlinear and inelastic behaviour of constituent materials and complex interaction of failure modes, analysis is difficult [38]. However, it is possible to predict failure modes by stress analysis with carefully chosen failure criteria in critical regions [39]. Numerical investigation of sandwich structures is usually reliable but the prediction of failure proves still to be difficult [40].

If the core is made from foam, the strength of core material and de-bond strength at the core skin interface almost entirely dictate the performance of sandwich panel under flexure [37]. The core is one of the weakest components of the sandwich panel and it can fail before faceplates.

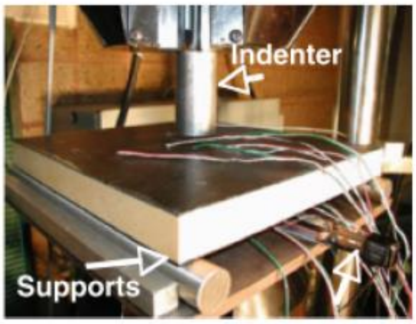
There are various types of failure modes for sandwich panels such as de-bonding of core face interface, compressive and tensile failure of the faceplates, face wrinkling, core failure, global buckling. As said before, it is possible that one failure mode can trigger or interacts with another failure mode.

2.4.1 Indentation & Impact

Indentation can hamper the load-carrying capacity of sandwich panels. Indentation is a result of impact loading or patch loading which results in crushing or local compression of core followed by local faceplate bending. This will result in a reduction in effective stiffness of the sandwich panel and lead to failure of the structure. Indentation under load can be prevented by choosing the appropriate core with high transverse stiffness.



(a) Delamination failure mode after impact.



(b) Impact test setup.

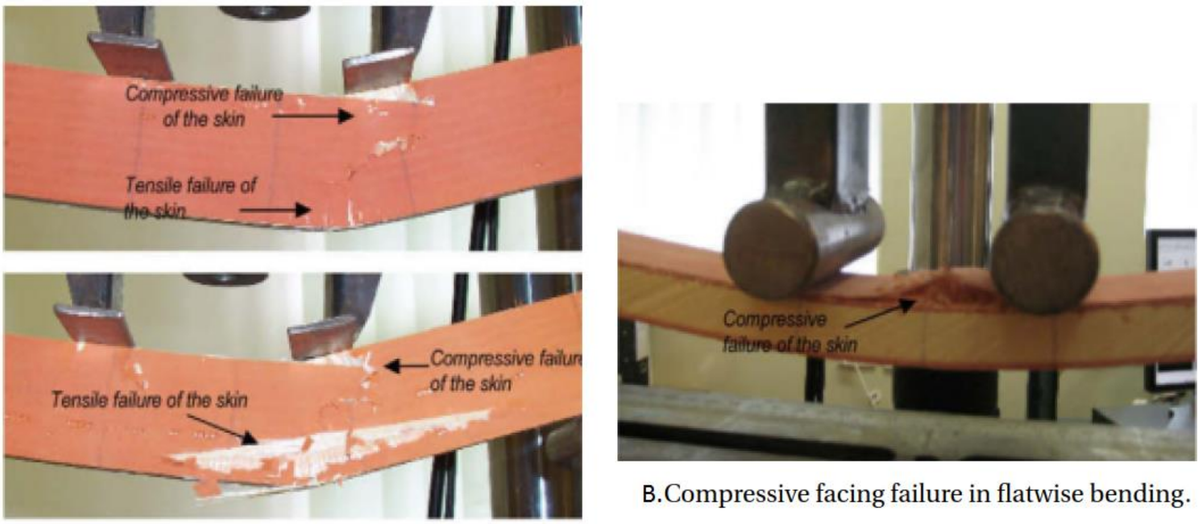
Figure 2.10 Indentation and Impact Failure [38]

For sandwich panels, indentation or contact law may be defined as the relationship between load introduction and deflection under that load. It plays a crucial role in sandwich panels with compressible cores under both quasi-static and impact loading [41]. These localised loads may induce thoughtthickness normal stresses and induce high loads in the core material and core-facing interface [40]. This is in contrast with core with high transverse stiffness that prevents indentation under load. Local rigidity of beam has a huge impact on the load-deflection relationship of the beam under load.

It is often difficult to spot impact damage with naked eyes, or damages might not be seen on an outer surface of a structure, but this does not mean that there are no effects on residual structural mechanical properties. If impact leads to plastic deformation, the structure’s stiffness will be reduced and it will be more susceptible to buckling under in-plane compression [38].

2.4.2 Face Sheet Compressive Failure

This failure occurs when the sandwich is subjected to pure in-plane bending or bending and low shear with a core of sufficiently high stiffness in the through-thickness direction [38][39].
This kind of failure can also occur when sandwich is subjected to edgewise compression. The core should have sufficiently high stiffness so faceplates are stable enough for a sandwich to fail under compression. Figure 2.11 gives an example of compressive faceplate failure in flatwise and edgewise bending. This failure type can be predicted by using maximum stress or yield strength criteria of face material.



A. Compressive and tensile facing failure in edgewise bending.

B. Compressive facing failure in flatwise bending.

Figure 2.11 Faceplate failure in flatwise and edgewise bending [42]

Generally, sandwich panels have lower compressive strength than tensile strength and therefore it is more prone to compressive failure. This is not true for composite laminates with randomly ordered fibre mat facing [43]. Manalo argues that core material having high strength has a significant contribution in shear and flexural stiffness and it should be considered to determine overall behaviour of composite sandwich beams [42]. If the core is not stiff enough in through-thickness direction then specimen will fail in faceplate wrinkling.

In conclusion, failure of faceplate due to compression occurs when the sandwich panel, with a core of sufficiently high stiffness in through-thickness direction, is subjected to edgewise compression or bending or bending & low shear. The core should be stiff enough such that faceplates are stable until their compressive strength is reached.

2.4.3 Face Core Debonding

Plate core debonding, also known as faceplate debonding, denotes faceplate-core interface failure. Interface failure is influenced by fracture toughness of interface i.e. by type and properties of the material used. Two failure criteria commonly used for prediction of this failure mode are inter-face fracture toughness or maximum shear strength of adhesive (when face and core are bonded by adhesive) [38].

(a) Debonding of facing after edgewise compressive load.

(b) Debonding of facing in three point bending test.

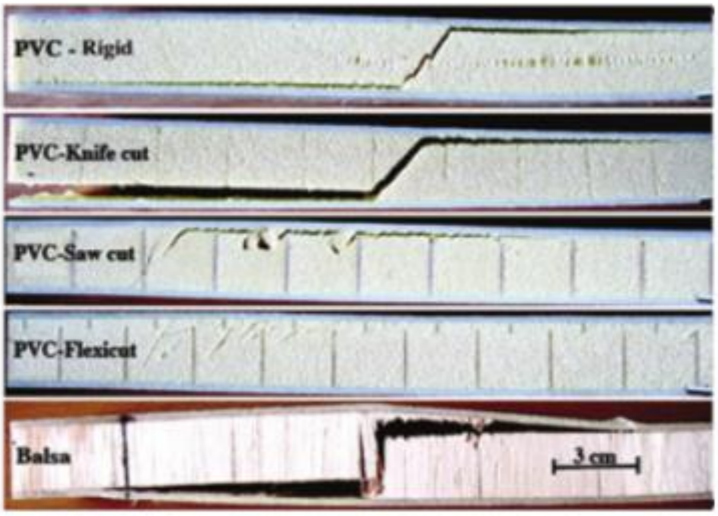
Figure 2.12 Face-core debonding under three-point bending and edgewise compressive load [44]

The sandwich panel having large initial cracks in an interface is more prone to face core debonding failure. This failure can also occur when preceded by another mode of failures such as faceplate yielding, faceplate wrinkling and core shear failure [45]. Daniel confirmed this and added that it is also likely to occur under or after impact loading [38]. Later it was concluded that crack propagation can occur in the faceplate-core interface for the core with higher density or in the core for lower core densities. Face core debonding of sandwich panel decreases stiffens of structure and makes it more vulnerable to buckling under compression as proven by Triantafillou et al. [45].

In conclusion, face core debonding or interface failure occurs after initial damage or a different type of failure mode. Interface fracture toughness or maximum shear stress of adhesive can be used for prediction of face core debonding.

2.4.4 Core Failure

Core material properties affect the performance of the sandwich panel. The main function of core in a sandwich panel is to carry shear loading, which is usually one of reason to use a sandwich structure. Failure of the core by shear is one of the common modes of failure in sandwich structures. Core failure can be observed in sandwich subjected to three/four-point bending or flatwise compression/tension. The core is primarily subjected to semi-uniform shear force under three/four-point bending or flatwise tension/compression and sandwich panel will fail when shear strength of the core is exceeded.



Core failure in four point bending test for different core types.

Figure 2.13 Core failure under four-point bending [46]

Distribution of shear stress and strain is uniform up to point until core material acts in its linear elastic state. When entering nonlinear/plastic range, yielding of core material occurs and shear stress and strain become highly non-uniform, peaking at the centre. Core failure is accelerated when shear and compressive stresses are combined. Yielding of core results in a reduction in stiffness and decrease in stability for faceplates, giving rise to other forms of failure such as faceplate wrinkling failure.

2.4.5 Buckling – Face Wrinkling

Buckling or wrinkling failure is often observed as a follow-up failure to initial facing, core, impact or de-bonding failure due to loss in stiffness or support of initial failure. Mahfuz et al [44] state that buckling occurs when membrane strain energy is converted into strain energy of bending without any change of externally applied load. Bending stiffness of a structure is affected by membrane forces. Buckling will occur when compressive membrane forces are large enough to reduce bending stiffness to zero. Buckling can cause significant reduction in stiffness and compressive strength of composite structures and can trigger other failure modes [47].

Transverse shear effects of the core make it important for prediction of failure load. Prediction of buckling load will be very un-conservative if these effects are not taken into account. Von-Karman equation for plates can be used for prediction of global buckling load. The point at which out-of-plane deflection is non-zero will correspond to buckling load. Failure load depends on the stiffness of faceplate and core, the thickness of core and aspect ratio of plates. Buckling load of the sandwich panel is affected by properties of the core.

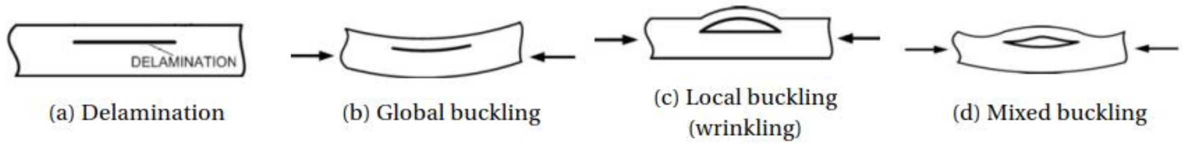


Figure 2.14 Delamination and possible subsequent buckling failure modes [47]

Global buckling of a composite sandwich structure only occurs if the core is sufficiently stiff enough in the through-thickness direction [38]. If not then face wrinkling might occur. In most of the research, face wrinkling is a typical type of failure mode observed in foam core sandwich panels. Face wrinkling is also known as local (short-wavelength) buckling.

Faceplate wrinkling will occur when the critical value of compressive stress of faceplate is reached. Value of critical stress is dependent on the modulus of elasticity of faceplates and core. Daniel [38] argues that the degradation of the core will result in a significant reduction in the value of this critical stress. Sandwich panels under pure bending or under compression are investigated, to understand their buckling or wrinkling behaviour. Critical buckling load, of columns under uniaxial edgewise compression, is dependent on thickness & stiffness of core. Dobyns [48] has discussed a lot of different models, their accuracy and applicability in his research "Correlation of Sandwich Face sheet Wrinkling Test Results with Several Analysis Methods".L.A. Carlsson and G.A. Kardomsteas have shown and explained various buckling modes of the sandwich panel in “Structural & Failure mechanics of Sandwich Composites”. The following figure shows various buckling modes explained by L.A. Carlsson and G.A. Kardomsteas. [8]

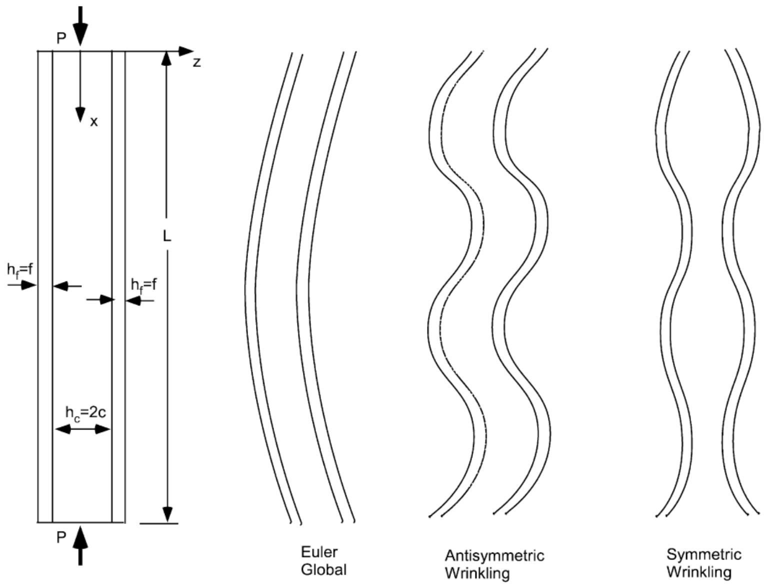


Figure 2.15 Buckling Modes [8]

2.5 Manufacturing

There are various processes for manufacturing of sandwich panel. Manufacturing of sandwich panel can be classified based on the process of bonding between faceplates and core. According to this process can be classified as ex-situ and in-situ bonding.

Ex-situ bonding:-

In the case of ex-situ bonding, the bond between faceplates and core is achieved by glueing with adhesive, by diffusion bonding or by brazing. The foam used in this method can be open cell or closed cell. Closed-cell foam can be used in this method when it is produced from aluminium alloys either by powder metallurgy route or by the liquid metal route. Aluminium, as well as other metals, can be used for manufacturing of open-cell foam core. Similarly, faceplates can be made from aluminium or any other metal such as steel.

In-situ bonding:-

In in-situ bonding, the core is closed cell foam. Metallurgical bonding is achieved between core and faceplates. This can be done in three ways. A foamable precursor is expanded between two faceplates. A metallurgical bond is established when faceplates and liquid foam comes in contact with each other. In this, it is difficult to realize that oxidation of both faceplates and core prevent forming a sound bonding. One more risk is that faceplate can melt. However, latter risk can be avoided by using a metal with a higher melting point, like steel plates used with aluminium core. In the case of steel faceplates and aluminium core, foaming of aluminium will not damage the faceplates.

In the second method, the surface of a foamable molten metal is rapidly/quickly solidified before it can foam into dense skin while the interior of metal evolves to a foam structure. This results in an integral type of foam structure. Magnesium alloys and aluminium alloys are used to make an integral foam sandwich. In this process material for faceplate and core is same.

In the third method, metal powders together with faceplates are compacted. In this method, the compacted assembly of the sandwich goes through various rolling steps so as to achieve the desired precursor and faceplate thickness. Afterwards, this three-layered composite is heated so that metal powders i.e. core layer is transformed into the foam. The melting point of faceplate material is higher than the melting point of foamable precursor material. Al-Si, Al-Si-Cu or Al-Si-Mg alloys are usually used for precursor composition while 3xxx, 5xxx and 6xxx series aluminium alloys are used for faceplates. The generalised chemical formula of aluminium alloys used can be written as $AlMg_xSi_y$ where x and y will change as per faceplate used in sandwich panel.

Manufacturing Process used by Havel:-

In general, the process is as follows,

1. Mould or rigid frame is created as per the required dimension of the sandwich panel. The dimension of mould will be derived from dimensions of the required sandwich.

2. In a rigid frame, sandwich materials are arranged properly i.e. metal faceplate (steel or aluminium) at the bottom, aluminium alloy used for foam spread evenly on the bottom faceplate and on top of this aluminium alloy another metal faceplate is placed.
3. This assembly is then put on a conveyer belt of the baking oven. If this sandwich assembly is put on a conveyer belt directly then during manufacturing or heating process due to softening of faceplate it may create a bond with a conveyer belt and will create a lot of problems. So to avoid this first graphite sheet is placed on a conveyer belt and on this assembly of sandwich is kept.
4. To generate confined condition this assembly is put under a certain apparatus. The main function of this is to apply pressure on top faceplate such as to overcome pressures which will be generated due to expansion of foam during manufacturing/heating process. Due to this confinement, the foam will spread in between the top and bottom faceplate and will result in uniform/even distribution of foam. This will result in a proper sandwich with correct structural integrity. If a confined condition is not created then top faceplate will move upward due to expansion of core which will cause uneven spreading of the core. This will also result in the non-uniform thickness of the sandwich panel. So the generation of confined conditions is necessary.
5. Baking process will continue for 45 minutes followed by 30 minutes of cooling. The cooling process is composed of both controlled cooling with machines and atmospheric cooling. After this sandwich panel can be taken out of the mould. Surface and edges of the panel can be softened and polished. The required sandwich is ready.
6. In the case of manufacturing of steel aluminium foam sandwich panel graphite sheet is not required in the process. Graphite sheet is not required because the steel plate is not going to create any adhesion or bonding with a conveyer belt. This is because steel can withstand high temperature.
7. Also, in the case of manufacturing of steel aluminium foam sandwich panel requirement of mould is reduced. In this case, side steel plates will be created with the width equal to the required thickness of the sandwich panel. These side steel plates will be welded on the bottom and top faceplate. This will perform the job of apparatus of creating confined condition and result in the manufacturing of a proper sandwich panel.

2.6 Finite Element Analysis

2.6.1 Introduction

In Engineering, finite element analysis is often used to solve problems through computational means. There are various finite element analysis programs that can be used in the analysis and design of the structure. Such programs are NASTRAN, SAP, NISA, ABAQUS and ANSYS which can solve very complex problems. An appropriate model with appropriate boundary condition and loading can be made in finite element software such that model will represent or replicates real scenario and problem. The geometry of the model will be defined along with elements which are used for analysis. Both solid and shell elements can be used in the analysis. Properties of elements and external conditions are defined as well. Finally, the proper meshing of the model will be done with an appropriate mesh size. Mesh size affects the accuracy and reliability of the solution. The fine mesh should be used to get accurate results. Modelling process should be done carefully otherwise if inputs are incorrect result from software will also be incorrect.

In thesis behaviour of the sandwich panel will be analysed with the help of finite element software. Finite element program used to model and analyse sandwich panels is ANSYS.

2.6.2 Shell Elements

Shell Elements can be used for 2D representation of element to be modelled. In a circumstance where two dimensions of structural elements are much greater than the third dimension and it is possible to neglect changes in analysed feature in the third direction, then shell elements can be used for modelling of a structural element. Use of shell element is sensible for static analysis of elements such as walls or slabs or any planar element as well as thin-walled elements. Influence of thickness on results is not considered in case of shell elements. This is due to the fact that the thickness of the modelled elements is much lower than the other two dimensions. The biggest advantage behind the use of shell elements is decreased in the computational time since the number of finite elements is reduced resulting in a reduction in equations to solve. The mathematical 2-D idealisation of 3-D structure is created with shell elements, which does not require split modelling thin dimension. This approach allows the use of efficient surface meshes. The geometry of the shell element can be rectangular or triangular. Each configuration has its own positive and negative effects. Triangular elements can be used in a lot of various shapes and geometries since it can fit perfectly in curved geometries as compared to rectangular elements. Higher node count can provide better approximation since at nodes analysis obtains values from degrees of freedom. Triangular elements can be made of 3 nodes at 3 corners or from 6 nodes 3 at the corner and an additional 3 at mid-point, latter one provides more accurate result than is 3 node equivalent. Similarly, rectangular elements can be 4-noded element or 8-noded element as shown in Figure 2.16. The 8-noded rectangular elements and 6-noded triangular elements are called higher-order elements.

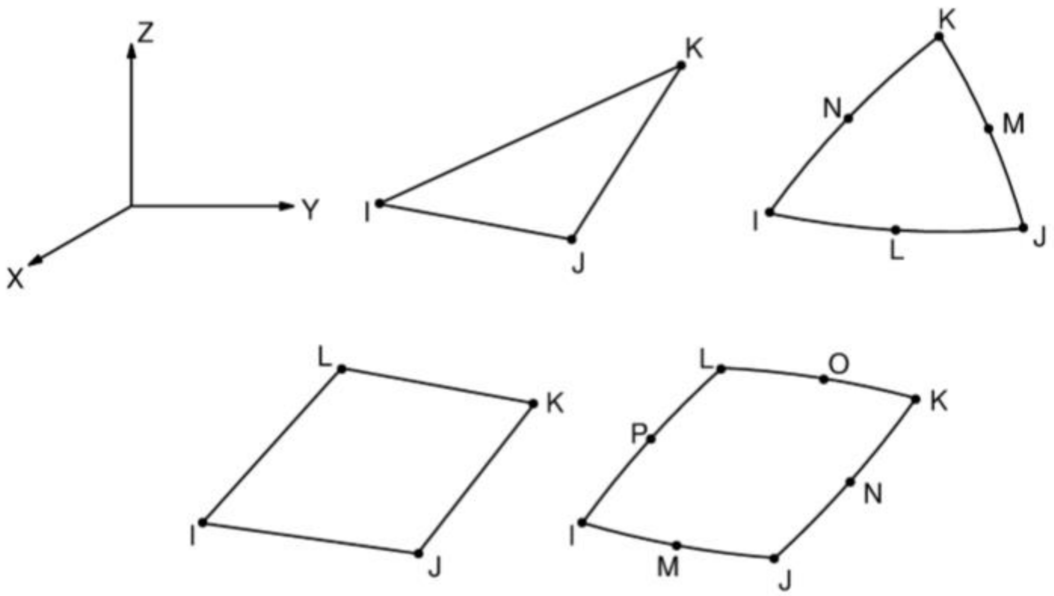


Figure 2.16 Shell Elements Triangle and Rectangle [9]

2.6.3 Solid Elements

In modelling, solid elements are used for 3D representation of the element. Accurate results can be obtained with the help of 3D model since it is closer to the real detail. Computationally solid elements are much more intensive. This due to the fact that for the same interpolation, a solid element has a higher number of nodes. Representation of thin-walled structure with the help of 3D solid element is conceptually simple. But the construction of efficient and accurate mesh with solid elements is difficult. Modelling thin structures using solid elements can be computationally very expensive. But solid elements offers a lot of advantages over shell elements. Variation of properties along thickness can be taken into account with solid elements. Solid elements can also provide stress variation along with the thickness of member which shell elements can't. Bricks and tetrahedral elements are used in solid modelling. Tetrahedral elements can be made of 4 nodes or 10 nodes which will be more accurate than its 3-noded equivalent. Similarly, brick elements can be 8-noded element or 20-noded element as shown in the following Figure 2.17.

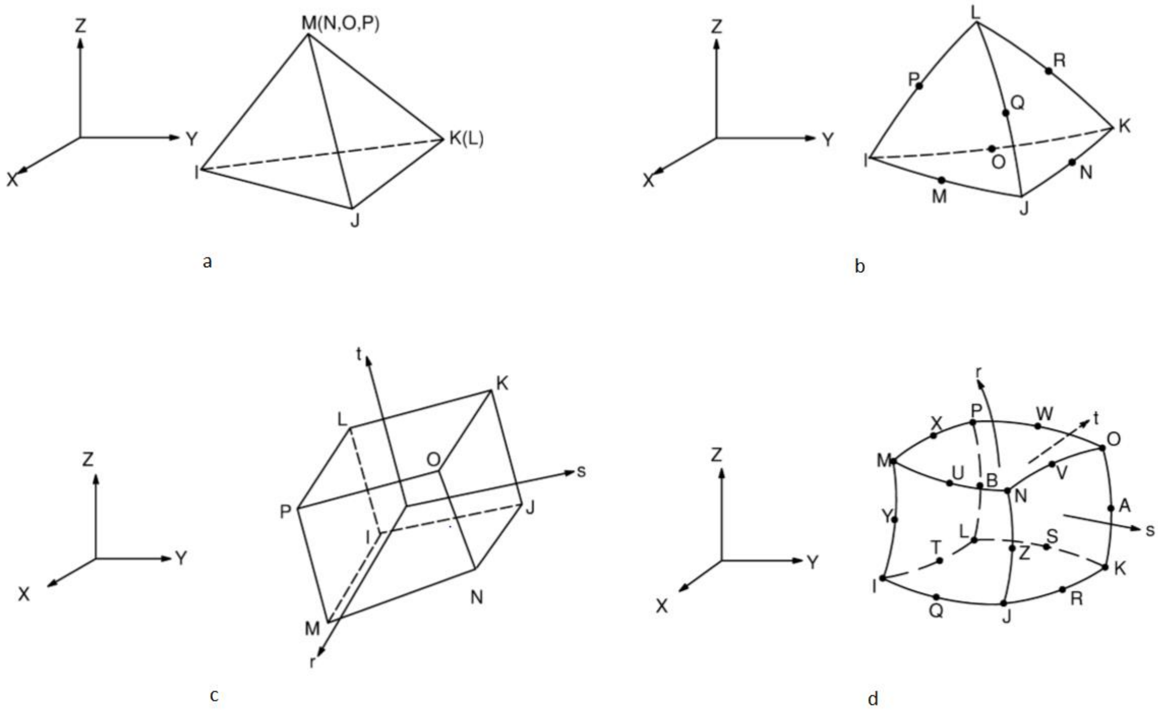


Figure 2.17 Solid Element Tetrahedral and Brick [9]

2.7 Buckling Theory

When the structure has high stiffness in one direction and low stiffness in other direction then it results in buckling of structure. When the compressive load in stiff direction is gradually increased structure collapses suddenly without any warning. Structures like a slender column or thin plate are vulnerable to buckling.

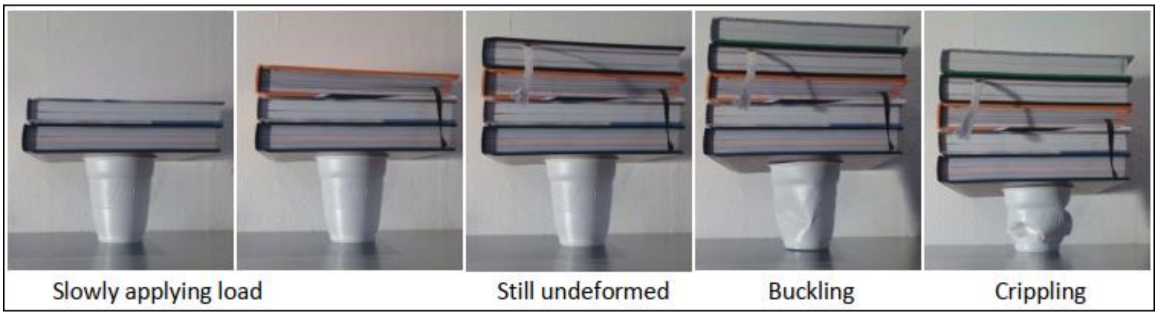


Figure 2.18 Simple example of shell buckling [10]

In the above example, the plastic/coffee cup is loaded in compression with gradually increasing load. Initially, when the load is small, no deformation is observed. But if the load is increased gradually then, suddenly at time't' cup will deform. This deformation is caused due to buckling. If the load is increased even further then finally cup will fail due to crippling. This gives a simple overall idea of buckling under compressive load. This example can be vaguely related to buckling of column or plate. The idea behind column or plate buckling is the same only its behaviour during buckling might change according to boundary conditions, type of loading, etc.

Buckling can be easily explained with the help of the load-displacement diagram. At the start, the structure behaves linearly along the primary load path. But at bifurcation point structure starts to follow the secondary load path. Bifurcation point is reached when the applied load is same as the buckling load. Stiffness of the secondary load path is lower than the primary path. This explains a sudden increase in deformation. In case of a column this secondary load path almost horizontal whereas for plate it moves up. The implication here is that plates can still carry some load after initial buckling, also called as post-buckling capacity. Refer to the diagram below,

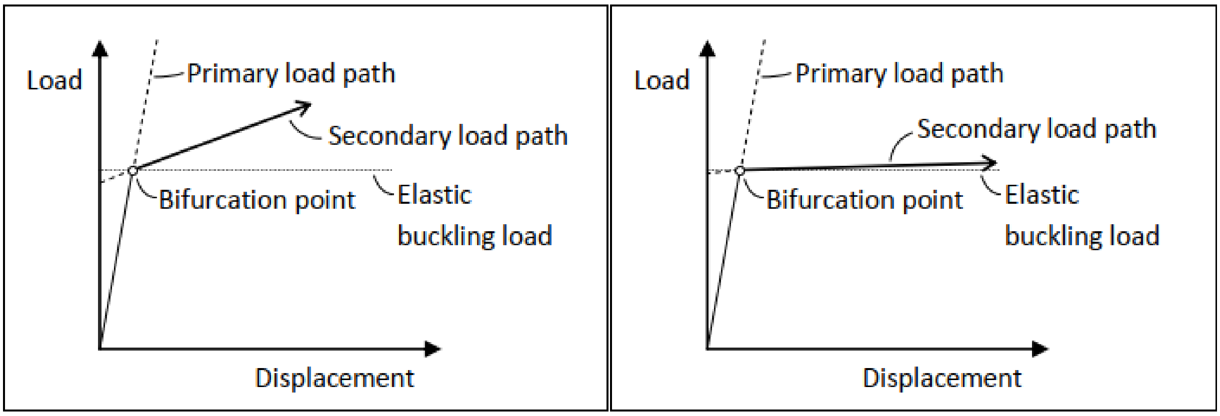


Figure 2.19 load-displacement diagram for elastic buckling (left: plate, right: column).[10]

2.7.1 Column Buckling

Buckling is an instability phenomenon, which is characterised by transverse deformation of the member under axial compression. Instability phenomenon is important for slender structures under compression. Therefore in steel structures, instability phenomenon is important. Theory of elastic stability is used to derive elastic critical load (Euler critical load), axial force or load at which an initially perfect elastic member may start exhibiting deformations that are not exclusively axial.

The resistance of a steel member subjected to axial compression depends on cross-section resistance or occurrence of instability phenomenon. Because of medium to high slenderness of steel member, instability phenomenon is generally a governing criterion in the design of steel members subjected to compression. The resistance of cross-section to axial compression depends on plastic capacity in compact sections, but also local buckling resistance through an effective elastic capacity should be taken into account. Relevant buckling mode and relevant imperfections in member should be taken into account to evaluate buckling resistance of member. Critical load in flexure buckling of a column can be given as,

$$N_{cr} = \frac{\pi^2 EI}{L^2} \tag{2.1}$$

Where EI is flexural stiffness about the relevant axis and L is effective buckling length. In case of the simply supported column, buckling length is equal to the actual length of the column. In other cases, buckling length differs according to support condition. This critical buckling load can be used to calculate slenderness of column, which then used to calculate the reduction factor. In case, when the slenderness of the column is lower than 0.2 reduction factor is unity, which means that full yield strength of member, can be used.

$$\lambda = \sqrt{\frac{Af_y}{N_{cr}}} \tag{2.2}$$

where, A is the cross-section area of the member.

f_y is the yield strength of the member.

λ is non-dimensional slenderness

The resistance of compressed members can be based on European design buckling cures. These five curves are the outcome of large scale numerical and experimental research that considers all imperfections in real compression member. Imperfections such as initial out of straightness i.e. bow imperfections, the eccentricity of load, residual stresses. Effect of imperfections are included by imperfection factor α

Reduction factor,

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda^2}} \tag{2.3}$$

where,

$$\varphi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] \tag{2.4}$$

φ is a constant based of slenderness & initial imperfection use to determine the reduction factor χ

Designed buckling resistance can be calculated as,

$$N_{b,Rd} = \frac{\chi\, A\, f_y}{\gamma_{M1}} \tag{2.5}$$

Euro code 1993 gives the requirement of steel design. Requirements are concerned with static strength and stability of steel structure. Euro code 1993-1-1 clause 6.3 gives method to calculate the buckling strength of the structure.

Compressive member should be verified against buckling as follows,

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0 \tag{2.6}$$

2.7.2 Plate Buckling

The plate is defined as a structural element where two of three dimensions is much larger than the third dimension. This is in contradiction to beam or column where only one of three dimensions in much larger than both others. Thin plates are very sensitive to buckling when loaded under compression. Due to the buckling of the plate, out of plane deformation will take place. Stress corresponding to buckling can be much lower than the yield strength of the material. The plate will not become instantly unstable when this deformation takes place since edges are still able to transfer loads. In contradiction to this, beam will become unstable at buckling load. For plate, if the load is increased furthermore, then deformation will increase. Total structure in which plate is functioning may become unstable because of the reduced load-carrying capacity of the plate, but for the plate itself, this might be a stable situation. But when buckling stress of plate becomes equal to yield of plate critical value of ratio b/t can be found. Critical buckling stress can be calculated with the Euler formula.

Euler buckling stress can be found out as,

$$\sigma_E = \frac{\pi^2 E}{12(1 - \upsilon^2)}\left(\frac{t}{b}\right)^2 \tag{2.7}$$

where,

E is the modulus of elasticity of plate in MPa

υ is the poison's ratio of plate

t is the thickness of the plate in mm

b is the width of the plate in mm

There are different methods to calculate the buckling strength of the plate. There is a reduced stress method and effective width method. Both of these methods are given in euro code 1993-1-5. For an unstiffened plate, principle directions need to be defined. Complete structure and way how it carries loadings to support determine principle directions. The longitudinal direction of the plate is chosen the same as the direction of the span of the complete structure. An aspect ratio of plate i.e. ratio of length to width can be used to calculate the number of half-sine 'm' at time of buckling.

To determine the reduction factor, plate slenderness can be used. Reduction factor can be used to calculate design yield strength of plate which is then used to determine buckling resistance of plate. In order to reduce the effect of plate buckling, plate slenderness should be kept at a minimum. If the plate slenderness is lower than 0.673 reduction factor is unity which means that full yield strength of plate can be used.

Slender plates have significant post-critical resistance. For plates with low aspect ratio a/b, post-critical resistance decreases gradually. This is because change in behaviour is observed, two-dimensional plate-like behaviour changes in a one-dimensional column-like behaviour which does not possess any post-critical resistance.For longitudinally stiffened plates with orthotropic properties this occurs for larger aspect ratio, aspect ratio a/b greater than 1 whereas for unstiffened plates this occurs for aspect ratio well below 1.

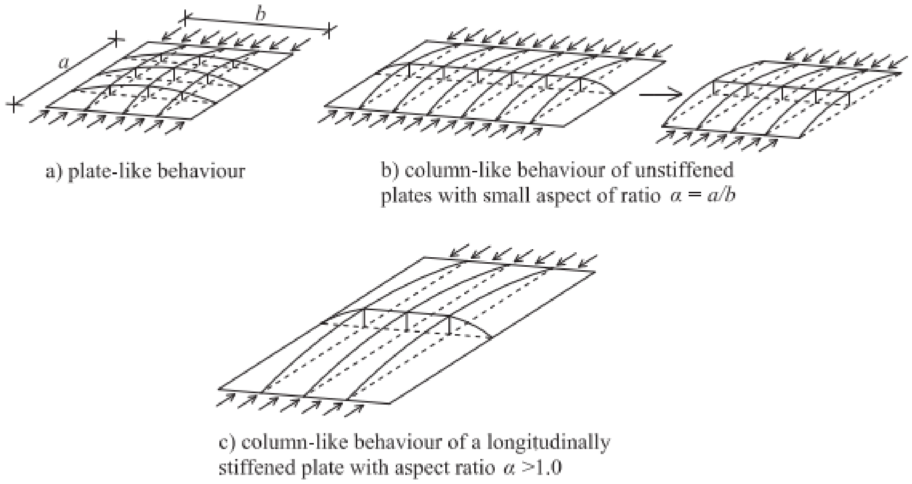


Figure 2.20 Plate-like and column-like buckling of the plate in compression [11]

In euro code 1993-1-5, in column-like buckling, plates are not supported along longitudinal edges. Therefore, critical stress for plate-like buckling will always be greater than critical stress for column-like buckling. Also for short plates, resistance depends on both plate-like and column-like buckling. Euro code 1993-1-5 gives a method for proper interpolation. In this chapter, plate-like buckling will be discussed.

When slender plates are subjected to compression it possesses significant post-critical buckling which can be utilised in the design of plated structures. Following Figure 2.21 shows common behaviour of slender plates subjected to compression

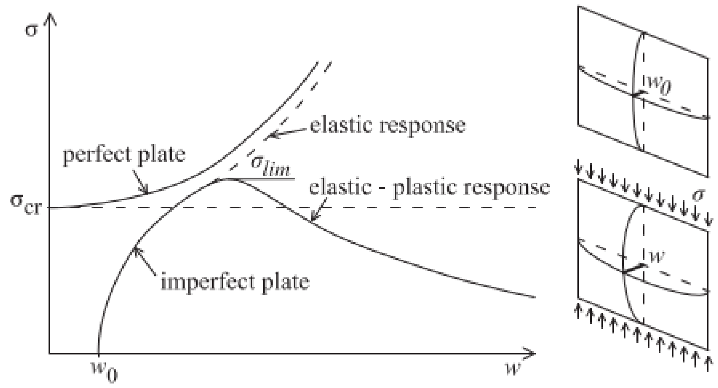


Figure 2.21 Post-critical response of slender plates subjected to compression [11]

Pre and post-critical behaviour for geometrically perfect plates are conspicuous but for plates with imperfections, gradual transition between pre and post-critical behaviour is observed. When plate possesses large imperfection, the behaviour is imperceptible. It is important that after reaching elastic critical stress σ_{cr} resistance is not exhausted, but it increases further until plastic collapse occurs. Redistribution of stresses takes place in the post-critical state, which results in stress reduction in the middle of the buckled part, where axial stiffness is decreased. This results in an increase of stresses near straight plate edges. When maximum edge stress reaches plate yield strength ultimate resistance is reached. In general, slender plates are not ductile enough to redistribute stresses by the development of plastic strains zones. It is not practical to deal with non-linear distribution of actual stresses σ_{act}. Therefore, two simplified methods are developed for practical design procedures.

The first method is called the reduced cross-section method or more commonly known as the effective width method. In this method in central buckled part of plate cross-section is reduced. In this case, we assume effective width b_{eff} adjacent to edges where stresses at edge equal to yield f_y of material overall effective width.

The second method is called the reduced stress method. In this average stress σ_{avg} of actual stress distribution σ_{act} in the ultimate limit state is considered.

Reduction of stresses or reduction of cross-section should be such that equilibrium is maintained with actual stress distribution.

$$P_{ult} = \int_0^b \sigma_{act} dx = b_{eff} f_y = b\sigma_{avg} = \rho b f_y \tag{2.8}$$

Therefore the reduction factor is,

$$\rho = \frac{b_{eff}}{b} = \frac{\sigma_{avg}}{f_y} \tag{2.9}$$

Both these methods result in the same result for any cross-section made up of plates/plated structure subjected under pure compression. In other cases, reduced stress method gives lower resistance than effective width method, since in reduced stress method design is governed by weakest plate element. Reduced stress method has advantage and disadvantage, the advantage is that its ease of application for more complex situations because it works on the level of stresses and disadvantage is that maximum strain in the plate is underestimated. Both effective width method and reduced stress method is given in section 4 and section10 of euro code 1993-1-5 respectively.

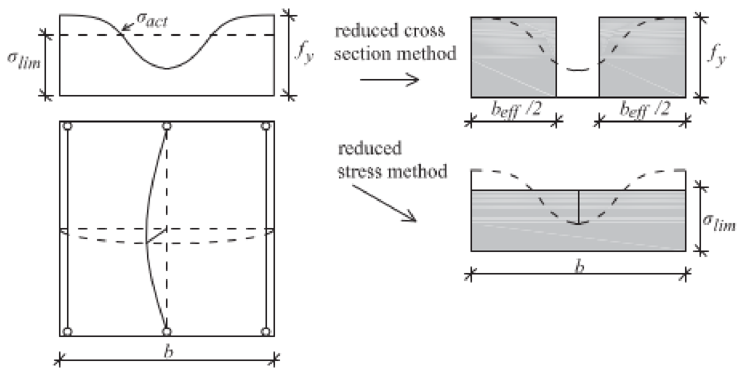


Figure 2.22 Basic idea of reduced stress method and reduced cross-section method [11]

Both methods reduced stress method and effective width method have similarities and also differences. The following list will give some differences,

Effective cross-section method

- Divide cross-section in parts
- Calculating the reduction factor for each part
- Multiply each part with its own reduction factor
- Used effective cross-section to calculate the resistance

Reduced stress method

- Divide cross-section in parts
- Calculate the reduction factor for each part
- Select lowest reduction factor
- Reduce yield strength based on the calculated reduction factor
- Calculate resistance based on reduced yield strength

Effective width method will be used in this thesis for calculation of buckling resistance of stiffened plate. The method is explained in Section 4.2.1

2.8 Considered Examples

2.8.1 Huisman structure

Application of sandwich panel in the design of heavy-lift structures with an aim to decrease weight without losing structural performance is incentive behind this thesis. This is because the fundamental demands of heavy-lift structures can be basically summarized as,

- High lifting capacity
- Reduced dead weight
- Optimized price and performance
- Low cycle fatigue

Heavy lift structure is a welded box girder type load-bearing structure with a hoist on aside. All major equipment is mounted inside box structure in an enclosed environment, protecting it from the harsh outside environment. There is no lattice-type structure around hoist crating open access.

Box structure provides an enclosed environment for the following equipment on the inside,

- Cabinets
- Winches
- Heave compensation cylinder
- Pressure vessels
- Tuggers
- Auxiliary equipment

Inside of box structure will be provided with cage ladders, stairs and platforms for safe and adequate access to equipment for operation, maintenance and service. There will also be doors and hatches to access platforms and walkways to equipment on the outside of box structure. The main access to structure is provided through the vessel.

The structure is divided into 5 parts with respect to height. 5 sections are,

- E-room Section at height h=0 mm
- Winch section at height h=9800 mm
- Lower heave section at height h=21700 mm
- Upper heave section at height h=38200 mm
- the top section at height h=50100 mm

All these sections are made from stiffened plate box girder. Details, dimensions and design of stiffened plate vary according to height. In this case, the heave section is considered arbitrarily.

Heave section is a box girder structure which starts at a height of 21700 mm. walls of box girder are made up of stiffened plates. In addition, transverse ring stiffeners are provided at the specified interval as per structural requirement.

In this thesis, it will be tried to replace the stiffened plate with the sandwich panel with the same dimension. So two dimensions i.e. the length and width of the sandwich panel will be the same as the stiffened plate. Only variable is the thickness of panel i.e. thickness of faceplates and thickness of the core. These thicknesses will be varied in order to achieve required strength i.e. strength to compete and replace the stiffened plate.

For this purpose, a stiffened plate from heave section of Huisman structure is considered. This plate is highlighted in Figure 2.23 & Figure 2.24. The lower part of Huisman structure which consists of E-room and Draw works is more dominant in fatigue. In the middle part of the structure, i.e. heave section compression is more dominant. To avoid fatigue, e-room & draw works is not considered instead the heave section is chosen. The heave section is made with stiffened plates arranged in an octagonal manner. From these stiffened plates, a plate with the highest dimensions is chosen. Highest dimensions results in more slender, which gives critical design & behaviour. Therefore, stiffened plate 5500x4320mm is chosen for the study. This stiffened plate is shown in Figure 2.23 & Figure 2.24. Since compression is dominant in heave section buckling analysis of stiffened plate is done. Based on this analysis appropriate sandwich panel will be designed which can replace this stiffened plate and will result in weight reduction.

FIGURE

CONFIDENTIAL

Figure 2.23 Huisman structure elevation [13]

FIGURE

CONFIDENTIAL

Figure 2.24 Huisman structure plan [13]

2.8.2 Specimen by Havel

In order to replace the stiffened plate by sandwich, the first step will be to understand the behaviour of the sandwich panel under axial compression. For this purpose, we will consider a sandwich specimen, which is considered by Havel during the test to do buckling analysis. Dimensions of the test specimen are length 1200 mm, width 80 mm and the combined thickness of 16 mm. This 16 mm thickness consists of, 2 mm thick faceplates and 12 mm thick core. The faceplates are manufactured with the steel of grade S235 with the modulus of elasticity of 210000 MPa, poisons ration of 0.3, the density of 7850 Kg per cubic meters. The core is made of aluminium foam with the modulus of elasticity of 500 MPa, poisons ration of 0.3, the density of 700 Kg per cubic meters.

Consider the sandwich panel with configuration as follows,

- Outer plates or faceplates made of steel with the yield strength of 235 MPa, modulus of elasticity 210000 MPa and Poisson's ratio 0.3
- Core made of aluminium foam with a yield strength of 5 MPa, modulus of elasticity 500 MPa and Poisson's ratio 0.3
- Length 1200 mm, width 80 mm and thickness 16 mm. This thickness is composed of 2 mm thick faceplates and 12 mm thick core.

These considered dimensions are the same as dimensions of test specimen used by German company Havel Metals for experimental purpose.

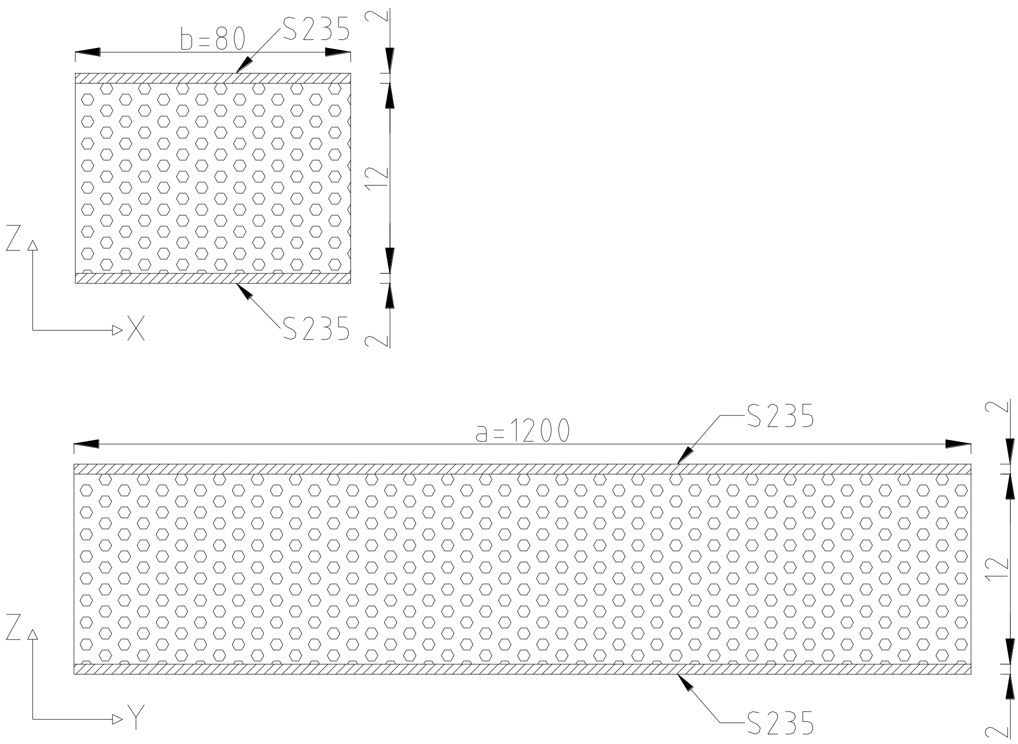


Figure 2.25 Sandwich panel used in the report by Havel [14]

3 Sandwich Panels - Application of Column Buckling Theory

For analytical analysis, column buckling theory is used. For purpose of analysis let us consider sandwich with same configuration as the one which is used by Havel in buckling experiment. In this chapter, attempts will be made to understand how different parameter will affect the behaviour of sandwich thereby affecting its buckling resistance. Parameters taken into account are, global geometric imperfection such as bow imperfections, local geometric imperfections, various material models. For steel two material models namely bilinear and multilinear are considered. Whereas for aluminium foam bilinear material model is considered.

To understand the buckling behaviour of sandwich panel, finite element model analysis in ANSYS is performed and results are compared.

3.1 Assumptions

- The load is applied only on flanges, not on the core
- Core does not contribute to buckling capacity (load carrying capacity) of sandwich panel
- The core has isotropic behaviour
- The density of the core is the same everywhere
- During manufacturing process, proper metallurgical bond is established between core and faceplates

3.2 Calculation of Buckling Resistance

According to assumption, only flanges are contributing in resistance and load-bearing capacity of sandwich panel. Therefore,

Moment of Inertia will be,

$$I = \frac{b}{12}(t^3 - t_c^3) \tag{3.1}$$

Buckling stiffness of sandwich will be,

$$EI = E_f * \frac{b}{12}(t^3 - t_c^3) \tag{3.2}$$

Euler elastic critical buckling load,

$$N_{cr} = \frac{\pi^2 EI}{L^2} \tag{3.3}$$

From this slenderness of sandwich can be calculated as follows,

$$\lambda = \sqrt{\frac{Af_y}{N_{cr}}} \tag{3.4}$$

where,

$$A = 2bt_f \tag{3.5}$$

$$t = t_c + 2t_f \tag{3.6}$$

Therefore the reduction factor is,

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda^2}} \tag{3.7}$$

where,

$$\varphi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] \tag{3.8}$$

b is the width of the sandwich panel

t_f is the thickness flange

t_c is the thickness core

t is the total thickness of the sandwich panel

L is the length of the sandwich panel

φ is value to determine the reduction factor χ

α is an imperfection factor

λ is non-dimensional slenderness

f_y is yield strength of faceplates

Therefore, as per column buckling theory, resistance of sandwich panel can be given as,

$$N_{bRd} = \chi \, A \, f_y \tag{3.9}$$

Bucking analysis is performed on the sandwich panel considered in Havel. Base on the assumption that only flanges contribute to buckling strength, bending stiffness is calculated which comes out to be $EI = 3.3 * 10^9 \; N - mm^2$. Since the panel is simply supported, buckling length is considered as 1200 mm. Based on this, the Euler elastic critical buckling load comes out to be 23 KN. For sample calculation, assume the sandwich panel has initial imperfections corresponding to the buckling curve ‘a’ then the value of imperfection factor α is 0.21. Therefore buckling resistance of sandwich panel comes out to be 20 KN. Please refer to Appendix A for in-depth calculation.

3.3 Material Models and Imperfections

Mechanical properties of the material can be illustrated with the help of the stress-strain curve. Stress-strain curve provides a graphical measure of strength and elasticity of a material. It predicts the behaviour of materials in use. Therefore, stress-strain curves are important.

In numerical, analytical and design models description of the entire stress-strain curve is important, especially when large plastic strains are encountered. Many stress-strain models are developed for steel. These either apply only to a limited range of strains or are too complex to be easily implemented in practice. There are various material models proposed for simplification of calculation. In this study, we will consider the bilinear model of steel, multilinear model of steel and bilinear model of aluminium foam.

Other than material models, geometric nonlinearity also has an effect on the behaviour of a structure. There are many ways to determine the theoretical elastic buckling load. For example, direct equilibrium method, virtual work method or via potential energy. These methods assume that structures are perfectly straight, without any initial deformation. However, in reality, structures are not perfect. After fabrication either by hot rolling or by welding, every structure like beam, column or plate will have slight initial imperfection like bow or twist. These imperfections will affect the behaviour of a structure. In this study, global and local imperfections are considered.

3.3.1 Material Model for Steel

Stress-strain curves are determined using unit stress and corresponding strain. Stress is obtained by load divided by the original cross-section area of specimen and strain is obtained by elongation divided by the original length. This curve is called the engineering stress-strain curve. But if the curve is obtained by using actual cross-section even after necking begins and by using instantaneous incremental strain, then curve is called as a true stress-strain curve.

In stress-strain curve straight-line relationship can be observed up to point called as proportional limit, this point coincides with the yield strength of the material. The ratio of stress to strain in the initial straight-line region is known as young’s modulus or modulus of elasticity E. In this region, loading and unloading results in no permanent deformation, therefore it is called as an elastic range. Flat long plateau for which stress is constant is known as the plastic range. The stress-strain curve also designates the ductility of the material. Ductility is defined as the amount of permanent strain (i.e. strain exceeding the permanent limit) up to point of fracture. Ductility can be measured from the tension test by determining percent elongation of specimen. Ductility permits local yielding due to high stresses, which allows stress distribution to change. Therefore, ductility is important.

Euro code 1993-1-5 gives four different material models for the FE model. Modes are shown below,

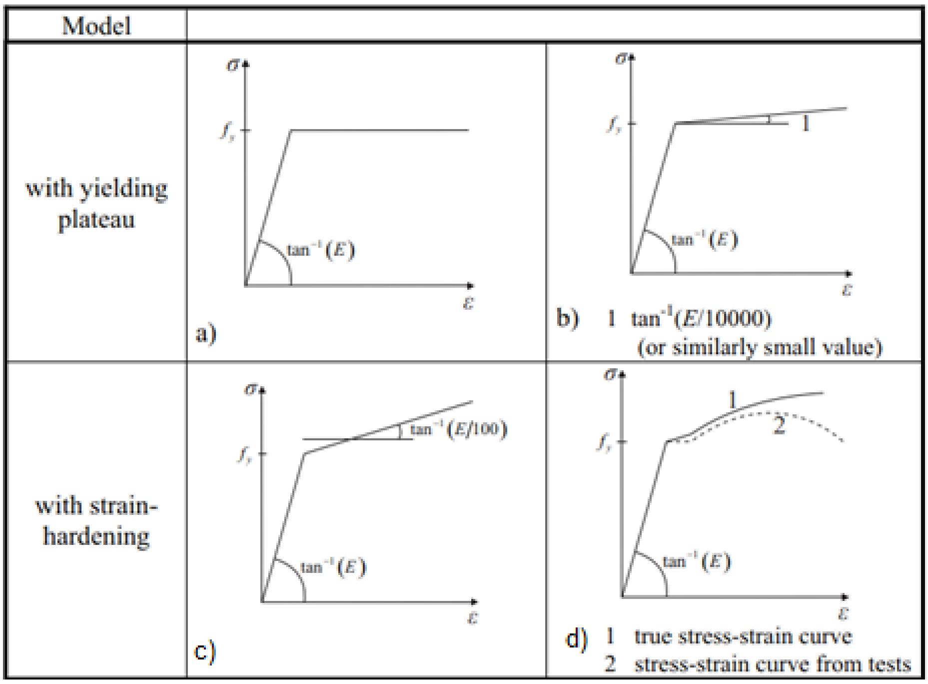


Figure 3.1 Modelling of material behaviour (from EN 1993-1-5) [49][11]

- In option ‘a’, strain hardening is not taken into account and yield plateau is horizontal. It is the easiest option.
- In option ‘b’, a very small slope (E/10000) is used for yield plateau. This option is a modification over option ‘a’ to avoid numerical instability.
- In option ‘c’, strain hardening is considered by using slope (E/100) and neglecting yield plateau
- Both options ‘d1’ & ‘d2’ represents the most realistic stress-strain curves. The only difference between these two options is the influence of reduced area during the tensile test.

Models used in this study for steel are, bilinear material mode which is option c and true stress-strain curve which is option d1 in above Figure 3.1. For finite element analysis, the true stress-strain curve developed by Huisman is considered which is illustrated in Figure 3.2. The bilinear material model used for steel is shown in Figure 3.3.

In true stress-strain curve, true stress and true strain are used for accurate definition of plastic behaviour of ductile materials by considering the actual or instantaneous dimensions. Therefore for the multilinear material model true stress-strain cure of steel is used. Also, to represent the elastic-plastic behaviour with a bilinear curve, the material model with strain hardening is used.

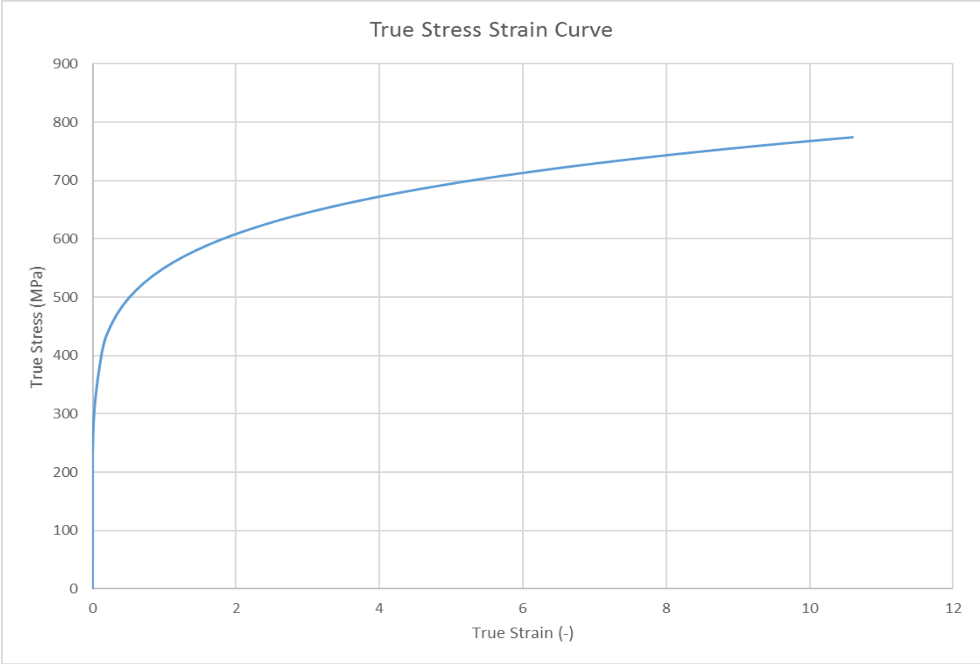


Figure 3.2 Steel material model Multi-linear

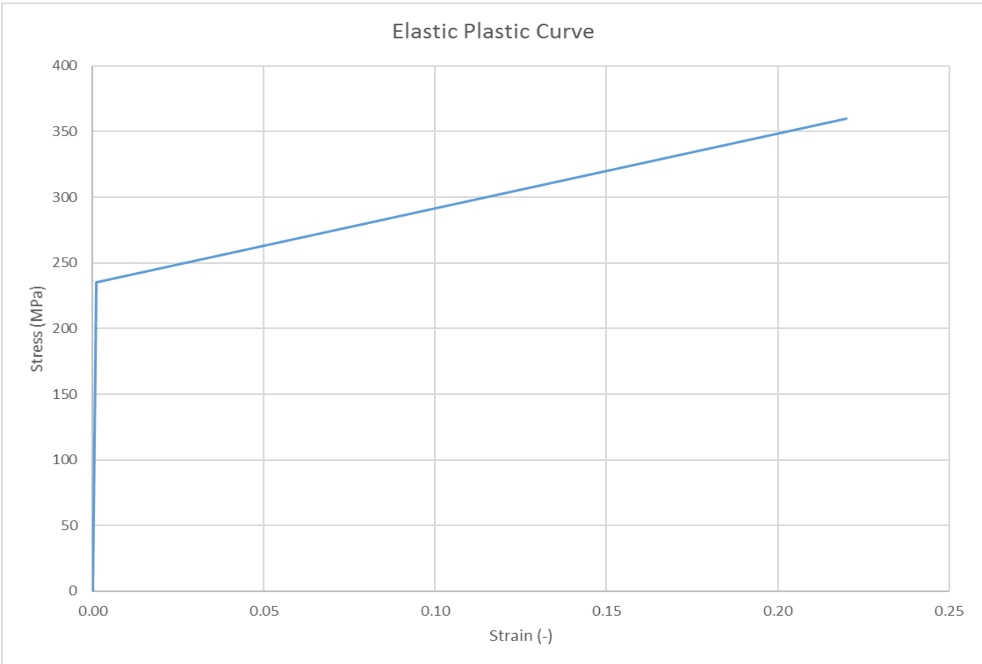


Figure 3.3 Steel material model Bi-linear

3.3.2 Material Model for Aluminum Foam

In the case of aluminium foam, only few material properties are known. Therefore, only bilinear material model of foam with no strain hardening is considered. Material that experiences no strain hardening during plastic deformation is an ideal plastic material. Option ‘a’ of Figure 3.1 will give an idea of the material. The material model used for aluminium is shown in Figure 3.4. It has linear elastic properties up to yield stress and then horizontal yield plateau. Elastic strain component is recovered if the material is unloaded after reaching some deformation strain. Reapplication of stress will result in retracing elastic line until yield stress is reached and then increasing plastic deformation. For aluminium foam since only few properties are known, it is not possible to make true stress-strain curve or curve with strain hardening. Therefore, the stress-strain curve with no strain hardening is used for aluminium foam.

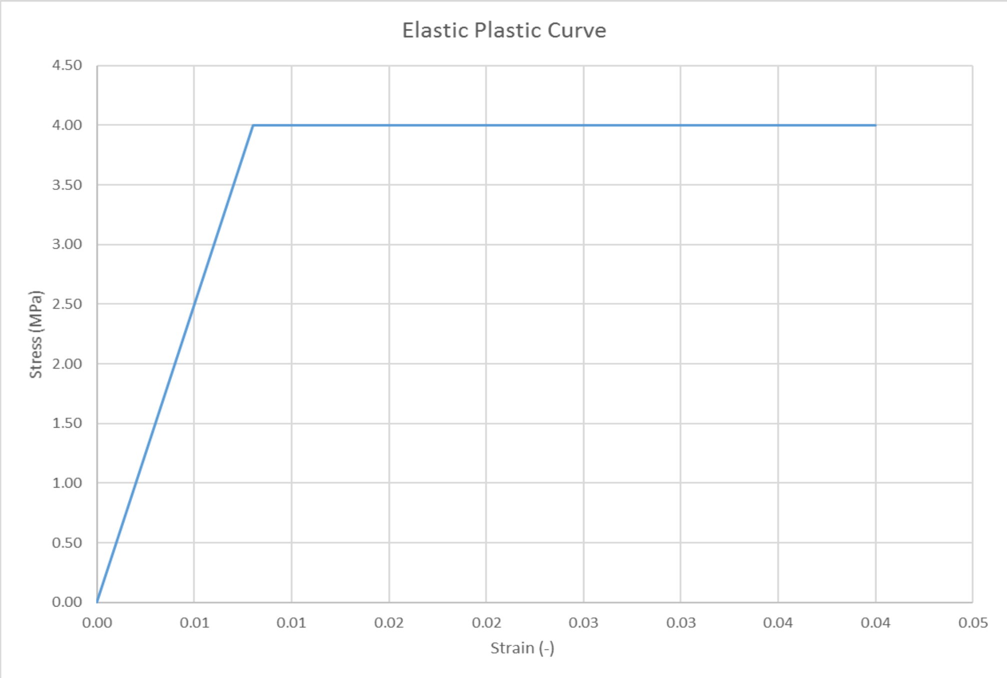


Figure 3.4 Aluminium material model Bi-linear

3.3.3 Geometric Imperfection

It is unlikely to find any prefabricated structural member in their original perfect geometry. Actual members always deviate from their original shape to a certain extent. Therefore, it is important to incorporate geometric imperfections with residual stresses in finite element analysis so as to stimulate true shape and structural behaviour of the test specimen. However, residual stresses are not considered in this study. Geometric imperfection arises because of fabrication and construction tolerances. These imperfections are modelled by introducing deformations in theoretical structures and by defining maximum amplitudes of these deformed shapes. Geometric imperfection method consisting of the increasing amplitude of geometric imperfections can be applied in order to cover residual stresses.

Presence of geometric imperfection seriously affects strength & behaviour of compressed plate elements and ignoring them in analysis results in unrealistic strength predictions. Compressive in-plane stress in faceplates may initiate buckling of the faceplate that may cause unstable behaviour thereby affecting structural integrity. Geometric imperfection can be global and local. Both local and global geometric imperfection has different effects on the stability of the structure. The separate and combined effect of both should be considered in the analysis. Adopted non-linearity analysis approach incorporates the effect of bow imperfection i.e. curved geometry of panel in the global sense as well as imperfections on the local level. When imperfection effect exists, additional local bending moments in sandwich panels particularly in faceplates are generated. In such cases, total stress is a result of stress due to axial load plus stress due to bending which is a result of imperfection.

Strength of the steel member is always sensitive to imperfection in the shape of its Eigen-modes. Buckling modes of the structure taken from an Eigen buckling analysis can be used as elementary imperfection shapes. An appropriate shape should be used for analysis. In ANSYS, initial deformation shapes can be easily imposed in shape of Eigen buckling modes with user-defined magnitude.

Therefore, to have a correct and accurate prediction of the buckling capacity of the sandwich panel it is important that proper material model and geometric imperfections are taken into account.

Global imperfection:-

In this case, four different models with different values of imperfection are considered. These imperfections are derived from or proportional to the length of the panel. This imperfection will give curved shaped geometry with initial deformation. The table shows values of imperfection imposed on the structure.

Model	Imperfection	Value of imperfection (Global)
a	L/300	4
b	L/250	4.8
c	L/200	6
d	L/150	8

Table 3.1 Design values of global geometric imperfection [12]

where, L is the length of the member.

The following figure can give an idea of structure with global geometric imperfection,

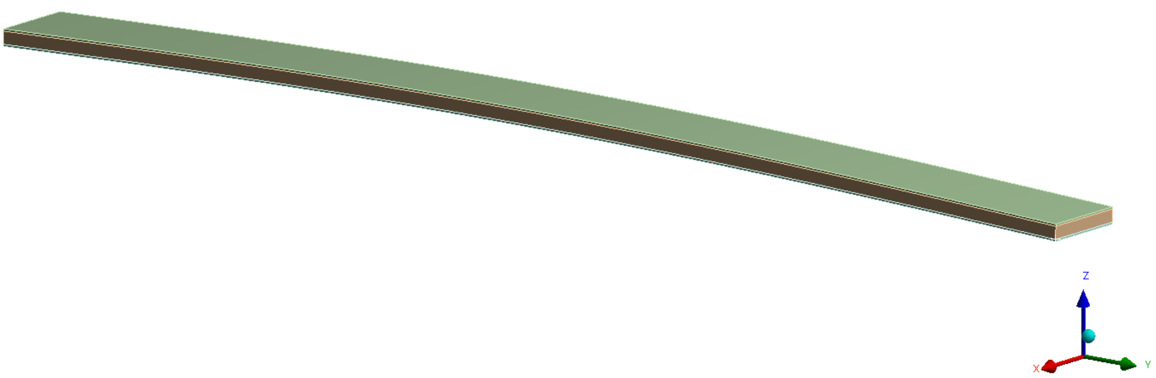


Figure 3.5 Global geometric imperfection

Local Imperfection:-

Along with global imperfection, local imperfections can also exit in structure. These imperfections are derived from or proportional to the length or width of the panel. Table 3.2 shows the values of imperfection imposed on the structure.

Type	Imperfection	Value of imperfection
Local	Min(a/200,b/200)	0.4

Table 3.2 Design value of local geometric imperfection [12]

where, a is the length of the member

b is the width of the member

The following figure can give an idea of structure with local geometric imperfection,

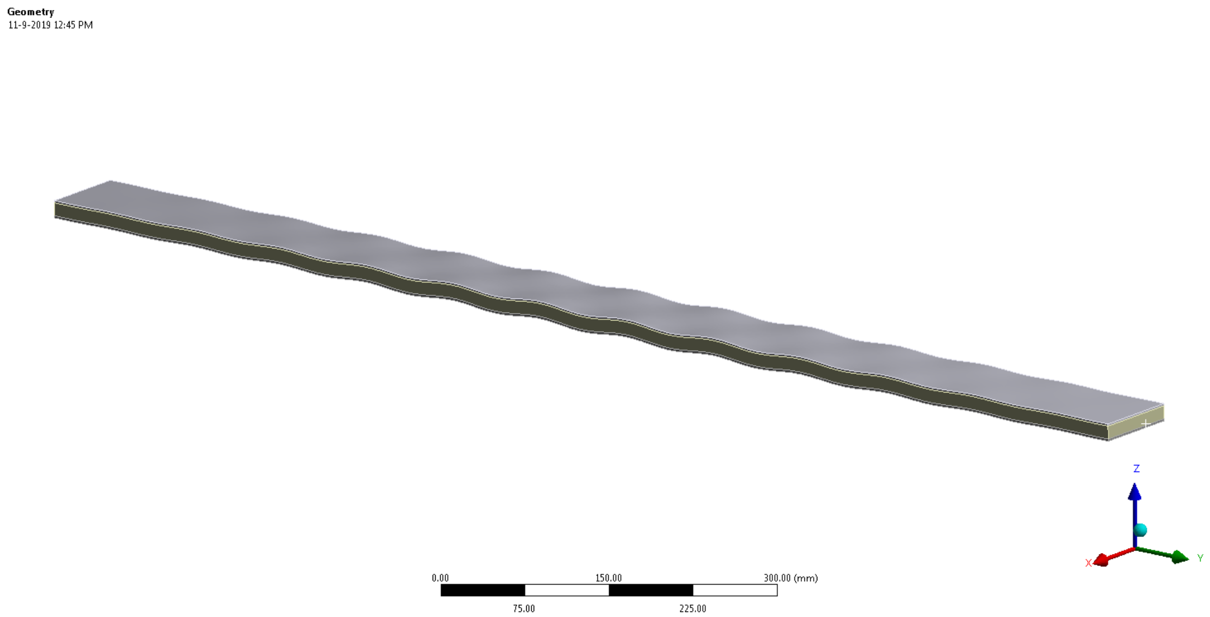


Figure 3.6 Local geometric imperfection

3.4 Effect on Resistance of Sandwich Panel

Finite element analysis considering geometric imperfections and material nonlinearity (models) is used to determine the resistance of sandwich panel under pure compression. For this purpose, steel aluminium foam sandwich panel is considered with the same dimensions as that of the specimen used by Havel. Eigen buckling analysis is done to get critical buckling load and buckling shapes. These buckling shapes will be used as initial imperfections in nonlinear analysis. The first fifty buckling shapes/modes are evaluated & plotted in the analysis.

As it is said in literature that, imperfections in the shape of buckling modes are critical. Mode 1 is the first Eigen buckling mode, which results in the global buckling of sandwich panel with 1 half sine. Mode 42 shows global buckling of sandwich panel with 42 half sines. Until mode 43, sandwich panel was showing global buckling with respective number of half sines. However, at mode 43 instead of buckling in 43 half sines sandwich started buckling on local scale i.e. local buckling of faceplates is observed. Therefore, mode 43 is the first mode, which gives local buckling or local imperfections in a sandwich panel. Also local imperfections observed in the faceplate are continuous or uniform. Therefore, mode 1 is first critical buckling mode which gives global buckling whereas mode 43 is first critical buckling mode which gives local buckling. Therefore, 1st buckling mode and 43rd buckling mode is used to impose global and local imperfections respectively.

For an in-depth explanation of finite element model such as loading, boundary condition please refer to Appendix A.

Global Geometric Imperfection:-

Buckling analysis is done on a sandwich with global geometric imperfection and various material models. These models are,

- Global Geometric Imperfections
- Global Geometric Imperfections with Steel Bi-Linear & Aluminium Linear
- Global Geometric Imperfections with Steel Bi-linear & Aluminium Bi-Linear
- Global Geometric Imperfections with Steel Multi- Linear & Aluminium Linear
- Global Geometric Imperfections with Steel Multi- Linear & Aluminium Bi-Linear

The following table shows load which can be applied when global geometric imperfections are considered along with various material models. Values of load are calculated with the help of finite element analysis.

Load corresponding to first Eigen buckling mode (Kilo-Newton)	Geometric Imperfection Model	Load in Kilo-Newton			
		Geometric Imperfections with Steel Bi-Linear & Aluminium Linear	Geometric Imperfections with Steel Bi-Linear & Aluminium Bi-Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Bi-Linear
20.9	a	9.8	9.8	10	10
	b	9.5	9.5	9.8	9.8
	c	9.3	9.3	9.6	9.6
	d	8.9	8.9	9.2	9.2

Table 3.3 Result of FE Analysis on a sandwich with global geometric imperfection and material models

To find the resistance of sandwich panel non-linear analysis of sandwich panel is performed. In this non-linear analysis rather than applying a load, displacement is applied and the resultant force is calculated. The displacement is applied in short increment until the point where it is not possible to achieve force convergence and model fails. The displacement increment & force reaction corresponding to it is arranged in tabular form. The maximum force reaction can be treated as the load-carrying capacity of the sandwich panel. In addition, to visualise this, graph between displacement increment and corresponding force reaction is plotted as follows,

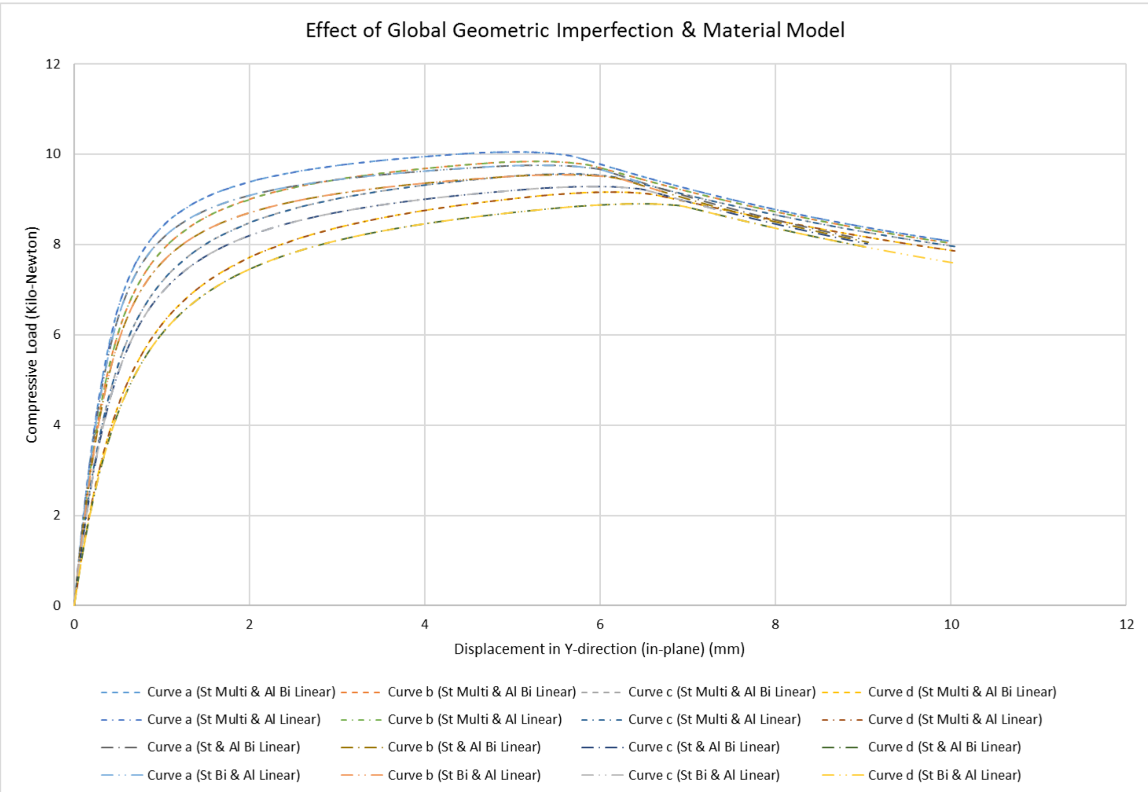


Figure 3.7 Graph between In-plane displacement vs Compressive Load (Global Imperfection)

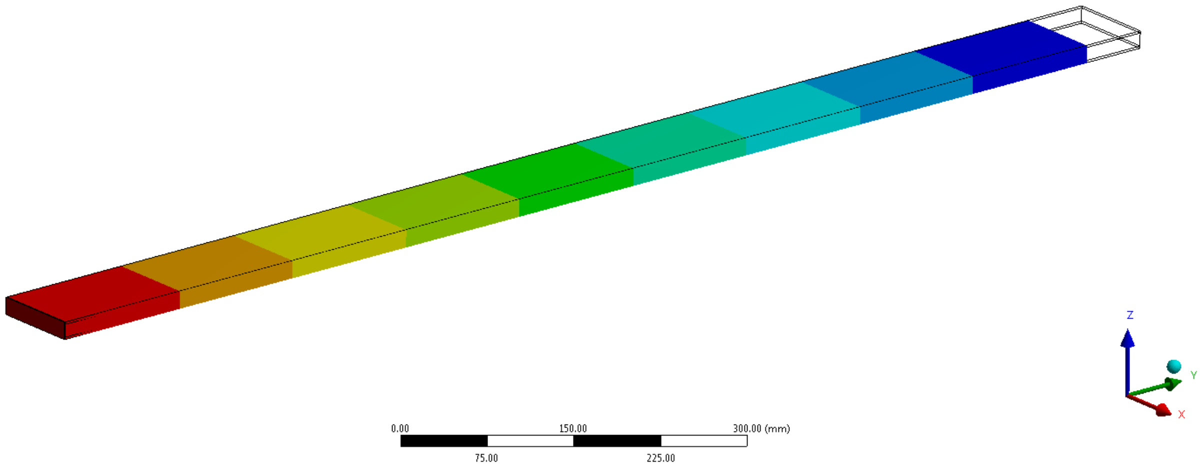


Figure 3.8 In-plane displacement of sandwich panel

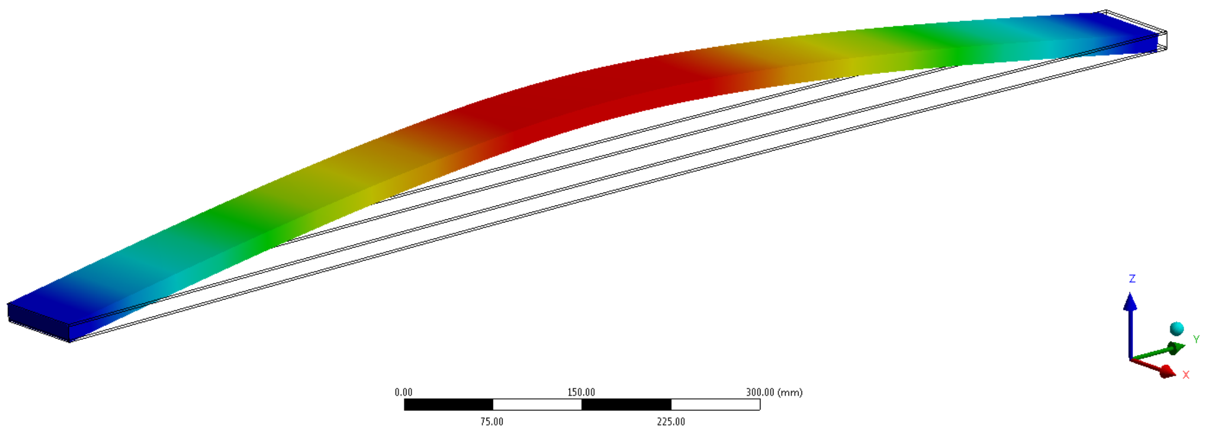


Figure 3.9 Out-of-plane displacement of sandwich panel

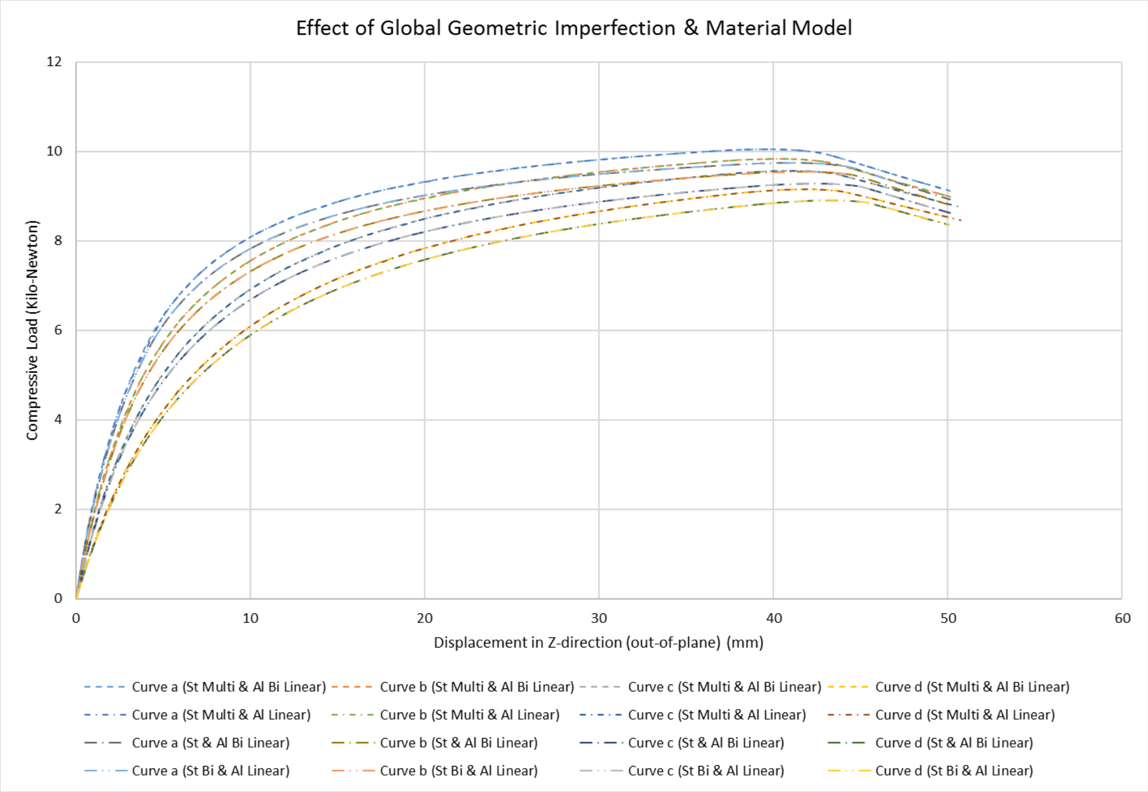


Figure 3.10 Graph between Out-plane displacement vs Compressive Load (Global Imperfection)

Local Geometric Imperfection:-

Buckling analysis is done on a sandwich with local geometric imperfection and various material models. These models are,

- Local Geometric Imperfections
- Local Geometric Imperfections with Steel Bi-Linear & Aluminium Linear
- Local Geometric Imperfections with Steel Bi-linear & Aluminium Bi-Linear
- Local Geometric Imperfections with Steel Multi- Linear & Aluminium Linear
- Local Geometric Imperfections with Steel Multi- Linear & Aluminium Bi-Linear

The following table shows load which can be applied when local geometric imperfections are considered along with various material models. Values of load are calculated with the help of finite element analysis.

Load in Kilo-Newton			
Geometric Imperfections with Steel Bi-Linear & Aluminium Linear	Geometric Imperfections with Steel Bi-Linear & Aluminium Bi-Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Bi-Linear
80	45	109	44

Table 3.4 Result of FE Analysis on a sandwich with local geometric imperfection and material models

In addition, to visualise this, graph between displacement increment and corresponding force reaction can be plotted as follows,

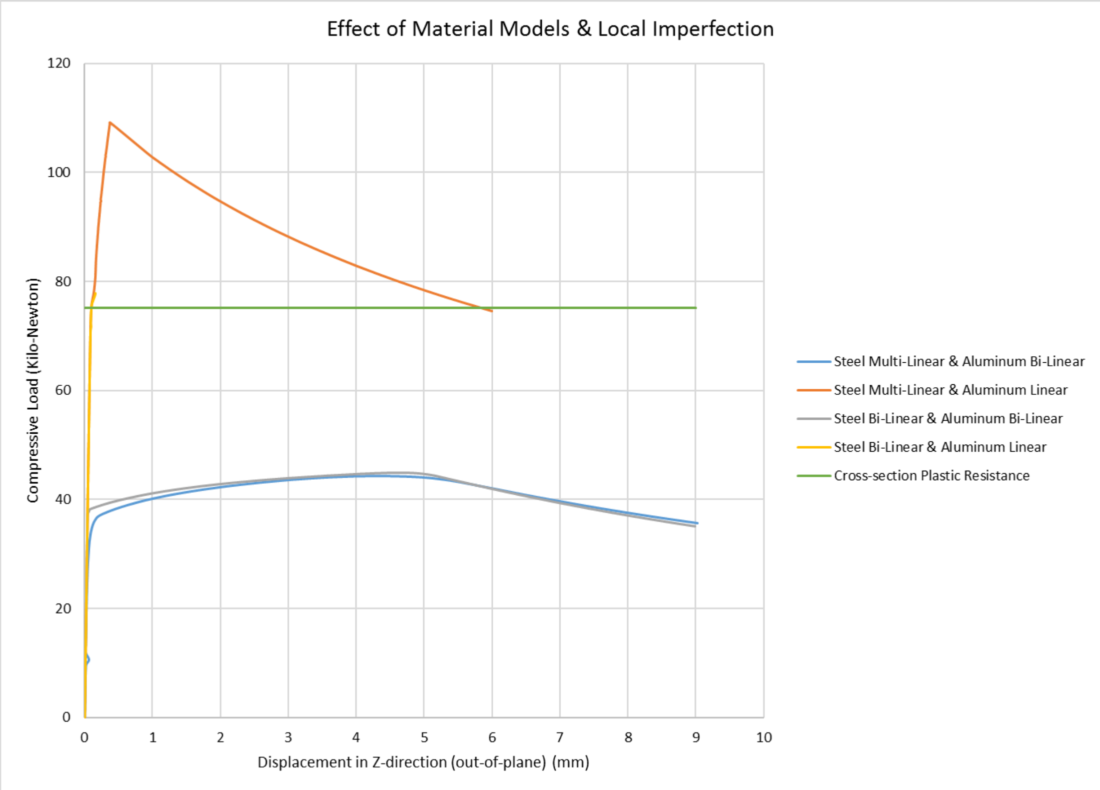


Figure 3.11 Graph between Out-plane displacement vs Compressive Load (Local Imperfection)

Combined Local & Global Imperfection:-

In the above cases, global and local imperfections are considered separately but in practice, global and local imperfections can occur simultaneously in structure. This might result in an additional reduction in the loading carrying capacity of the structure. This value of load can be calculated analytically with help of following formula,

$$\frac{F}{F_{buck,global}} + \frac{F}{F_{buck,local}} \leq 1 \qquad (3.10)$$

According to this formula, the load-carrying capacity of a sandwich panel with both local and global imperfections is calculated and arranged in the following table,

Load corresponding to first Eigen buckling mode (Kilo-Newton)	Geometric Imperfection Model	Load in Kilo-Newton			
		Geometric Imperfections with Steel Bi-Linear & Aluminium Linear	Geometric Imperfections with Steel Bi-Linear & Aluminium Bi-Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Bi-Linear
20.9	a	8.7	8.0	9.2	8.2
	b	8.5	7.9	9.0	8.1
	c	8.3	7.7	8.8	7.9
	d	8.0	7.4	8.5	7.6

Table 3.5 Result of FE Analysis on a sandwich with global & local geometric imperfection and material models

To understand the effect of various parameters on the buckling strength of the sandwich panel, different graphs can be drawn. These graphs will explain how the buckling strength of the panel will change if one parameter changed and all other parameters are kept constant.

1. Effect of thickness of faceplate:-
   Plotting graph between buckling resistance of sandwich with respect to the thickness of faceplate with all other parameters are constant. This graph shows the effect of thickness of faceplate on the resistance of the sandwich panel. The graph is as follows,

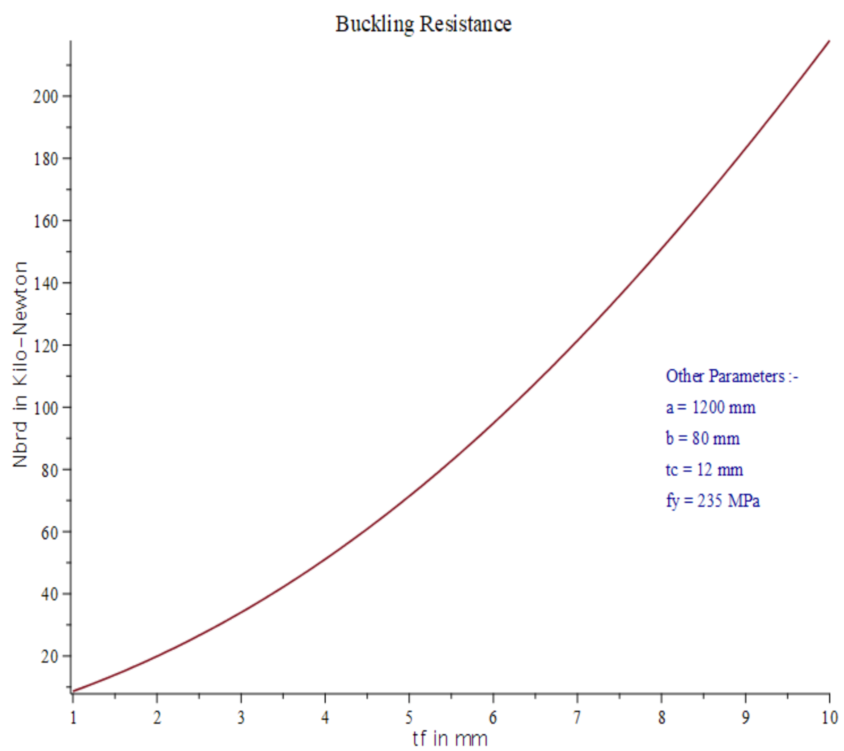


Figure 3.12 Effect of thickness of faceplate on the buckling load

From this graph, we can observe that buckling resistance of sandwich is proportional to the thickness of the faceplate i.e. increase in the thickness of faceplate results in an increase in resistance of the sandwich panel. At an initial point, an increase in the thickness of the faceplate by 1 mm results in an increase in buckling resistance by 20-kilo newton.

2. Effect of thickness of core:-
   Plotting graph between buckling resistance of sandwich with respect to the thickness of core with all other parameters are constant. This graph shows the effect of thickness of core on the resistance of the sandwich panel. The graph is as follows,

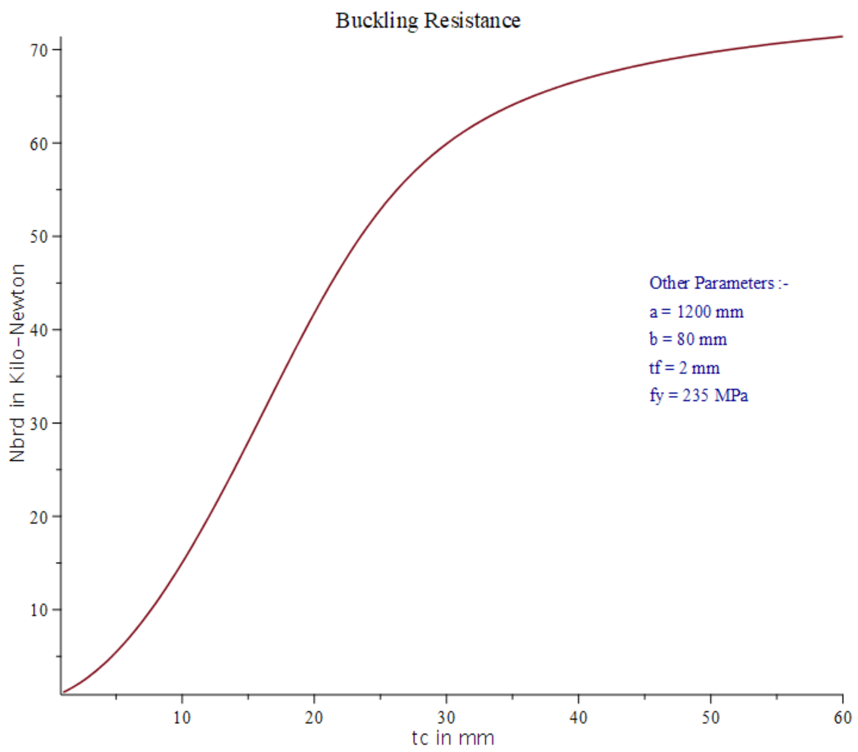


Figure 3.13 Effect of thickness of core on the buckling load

From this graph, it can be observed that an increase in the thickness of the core has a positive influence on the buckling resistance of the sandwich panel. This is due to the fact that the thickness of the core affects the moment of inertia of the sandwich. From equation 3.1, it can be observed that the thickness of the core will affect the moment of inertia of the panel. Increase in thickness of core will increase the distance between faceplates thereby increasing the resulting moment of inertia. So even though the thickness of core has not included in the equation of resistance since its contribution is neglected still it affects buckling resistance indirectly due to its effect on inertia. Therefore, it can be concluded that the thickness of the core has an indirect effect on the sandwich panel. Also, it can be observed that this effect is small as compared to the effect of faceplate thickness. For 1 mm increase in the thickness of the core, an increase in buckling resistance is really small.

In the above graphs, out of two parameters t_c & t_f one is always constant, so the effect observed is related to that specific value of another parameter. One more type of graph can be plotted where both parameters t_c & t_f of the sandwich panel are varied simultaneously.

3. Effect of thickness of core & faceplates:-
   This type of graph will explain the effect of thickness of flange and thickness of core on the resistance of sandwich simultaneously.

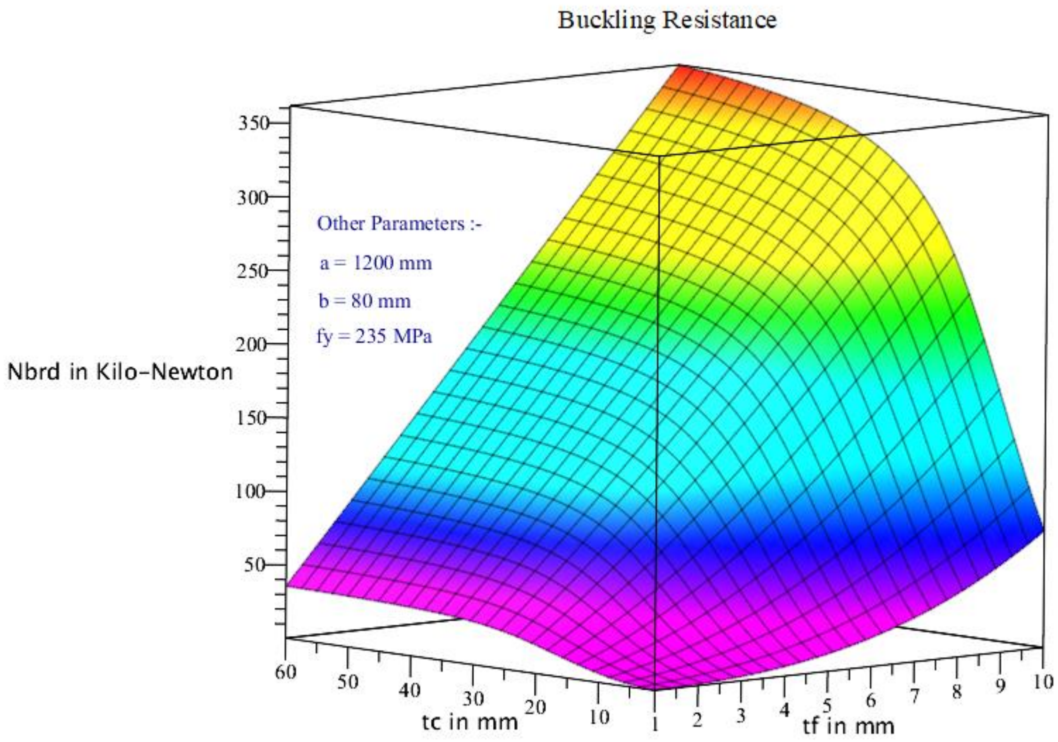


Figure 3.14 Effect of thickness of faceplate and core on the buckling load

Above graph illustrate the effect of thickness of core and faceplate on buckling resistance of sandwich panel simultaneously. From the slops of the graph, we can observe that the thickness of faceplates has a large impact than the thickness of core on buckling resistance. Generally, the minimum thickness of core is 10 mm and it can be increased up to 100 mm or even more as per requirement. This is because an increase in core thickness will have a small effect on the increase in weight. On contrary thickness on faceplates will vary with the minimum being 2 mm and maximum 10 mm. Since the density of faceplates is high and it has a high effect on the weight of a sandwich panel.

3.5 Comparison of Buckling Loads

Global Imperfections:-

Reduction factor can be used to compare the value of Eigen buckling load with load at buckling calculated with help of FEA. Reduction factor will illustrate a reduction in Eigen buckling load due to the presence of various geometric imperfections and considered material models.

Geometric Imperfection Model	Reduction in Buckling Load			
	Geometric Imperfections with Steel Bi-Linear & Aluminium Linear	Geometric Imperfections with Steel Bi-Linear & Aluminium Bi-Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Bi-Linear
a	0.47	0.47	0.48	0.48
b	0.46	0.46	0.47	0.47
c	0.44	0.44	0.46	0.46
d	0.43	0.43	0.44	0.44

Table 3.6 Reduction in buckling load for considered global geometric imperfection & material models

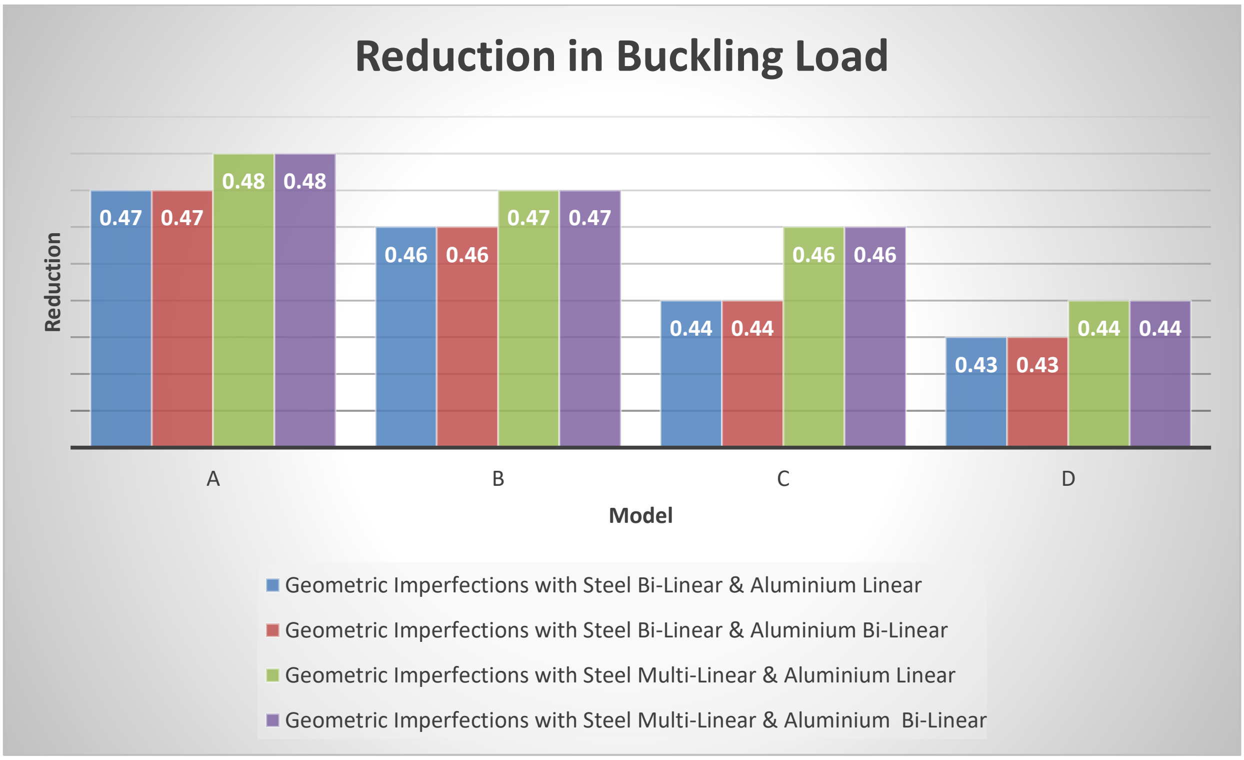


Figure 3.15 Graphical representation of reduction in buckling load for considered global geometric imperfection & material models

Combined Local & Global Imperfection:-

Geometric Imperfection Model	Reduction in Load			
	Geometric Imperfections with Steel Bi-Linear & Aluminium Linear	Geometric Imperfections with Steel Bi-Linear & Aluminium Bi-Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Linear	Geometric Imperfections with Steel Multi-Linear & Aluminium Bi-Linear
a	0.41	0.38	0.44	0.39
b	0.41	0.38	0.43	0.39
c	0.40	0.37	0.42	0.38
d	0.38	0.36	0.40	0.36

Table 3.7 Reduction in buckling load for considered global & local geometric imperfection and material models

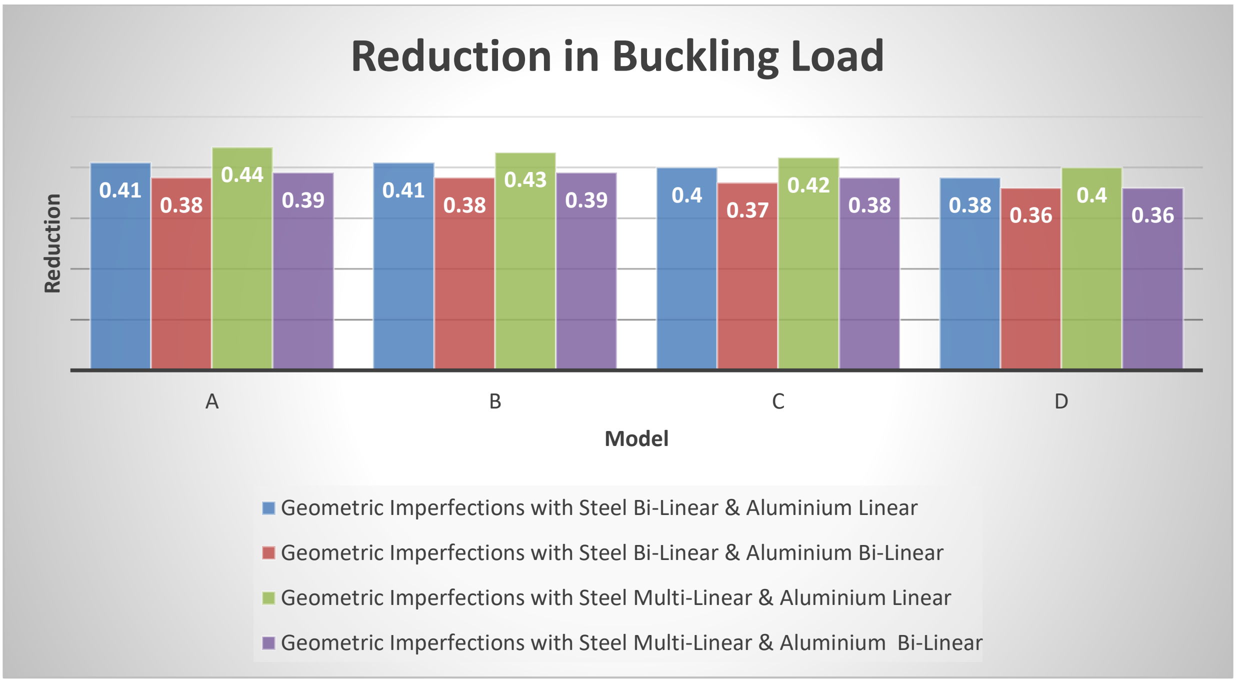


Figure 3.16 Graphical representation of reduction in buckling load for considered global & local geometric imperfection and material models

3.6 Havel Experiment

The sandwich panel is a quite new material and an uncommon structure. The sandwich panel can be used in designing of steel structures. In order to use an uncommon structure in design, it is important to have a proper understanding of these structures and their structural behaviour. For this purpose, column-buckling test on the sandwich panel is performed in the University of Surry.

For this test, steel aluminium foam sandwich panel is chosen. Steel aluminium foam sandwich panel consists of two thin steel plates, which are separated by low-density aluminium foam core. In this study, the buckling behaviour of steel aluminium foam sandwich is compared with the steel plate. The main aim behind the use of sandwich panel is to achieve high stiffness and weight reduction compared to the conventional structure. Therefore, dimensions of the sandwich panel and steel plate are chosen in such a way that both have the same weight.

Dimensions of both specimens are 1200mm by 80mm i.e. length is 1200 mm and 80 mm width. The thickness of the sandwich panel is 16 mm whereas the thickness of the steel plate is 5 mm. The sandwich panel consists of 2 mm thick S235 steel faceplates and 12 mm thick aluminium foam core. The following Figure 3.17, Figure 3.18, Figure 3.19 will give an idea of a sandwich panel & steel plate.

Figure 3.18 Steel plate cross-section

Figure 3.17 Sandwich cross-section

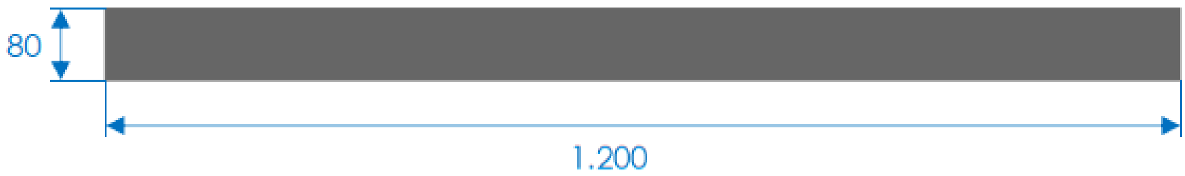


Figure 3.19 Plan of both specimens

Both test specimens are simply supported i.e. both ends have pinned support condition. Therefore, the buckling length of the sandwich panel is equal to physical length. Also, at support rotation is not restrained. In-plane compressive load is applied on test specimens. The load is applied in small increments and corresponding out of plane deflection is measured. Figure 3.20 will show a test apparatus and setup.

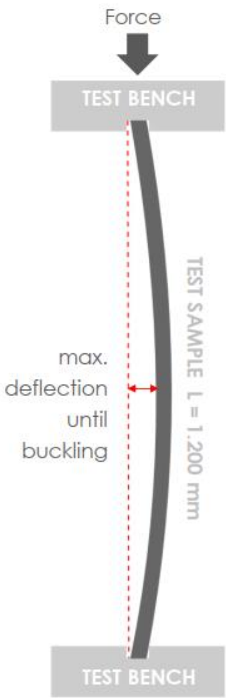


Figure 3.20 Test setup and apparatus of Havel experiment

From the test, it was observed that the sandwich panel fails at significantly higher load as compared to steel plate. Failure load for the sandwich panel was around 38.7 KN whereas failure load for the steel plate was around 1.4 KN. Therefore, the failure load for the sandwich panel is 27.6 times more than that of the steel plate. At failure load out of plane deformation for steel plate was 4.5 mm whereas for sandwich panel it was 1.5 mm. The graph between the applied load and corresponding displacement is plotted for steel plate and sandwich panel. Figure 3.21 will show both graphs.

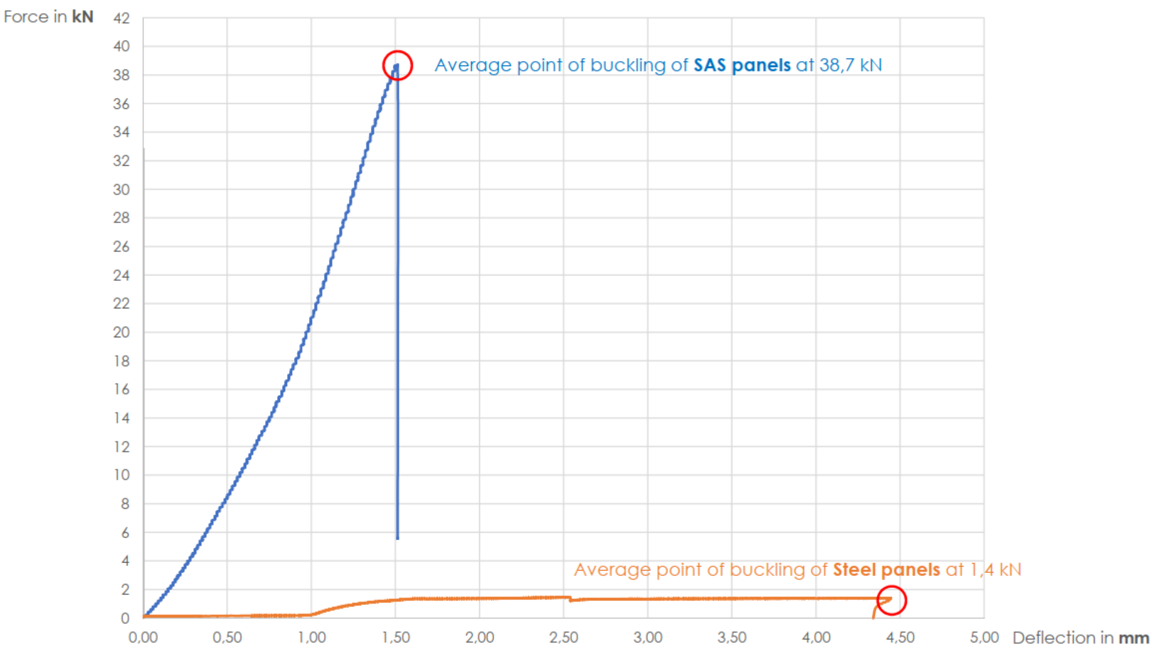


Figure 3.21. Graph between applied load and corresponding displacement

Observed buckling mode of the sandwich panel at the time of failure is shown in Figure 3.22,

Figure 3.22 Buckling mode of sandwich panel

Therefore, from an experiment, it can be concluded that the sandwich panel has high buckling capacity as compared to the steel plate of the same weight.

At the time of failure, the point of buckling of the sandwich panel is close to support. This is not an expected result. Point of buckling should be at the centre/middle of the sandwich panel. From the figure, we can observe that dial gauge or strain gauge is attached to the sandwich panel at centre. Because buckling was expected at the centre of the sandwich panel, instead it was observed near support. This behaviour might be due to one of the following reason,

- Density of foam was low at this point
- Foam might have a lot of voids at this point
- Support conditions were not proper
- Load is not applied properly
- Presence of Initial imperfections such as manufacturing imperfections or defects

Therefore, whether this experiment is a good example of buckling or not is debatable. Also, we might argue over the outcome or result of the experiment.

3.7 Results and Observations

- Column buckling theory can be used in prediction of the load-carrying capacity of a sandwich panel.
- Considered material models affect the load-carrying capacity of a sandwich panel.
- In the presence of geometric imperfections, material models used for faceplates have a significant impact on load-carrying capacity whereas material models of the core have a very small or negligible impact on the load-carrying capacity of sandwich panel. This might be due to the fact that in the global scale core has no contribution to buckling resistance of the sandwich panel. This also justifies theory, which states that the contribution of core to buckling stiffness is negligible. As per the theory, stiffness of sandwich panel can be calculated by the following formula,

$$EI = E_f * \frac{b}{12}(t^3 - t_c^3)$$

From this equation, it can be observed that stiffness of panel is dependent on the modulus of elasticity of faceplate. This can explain why the material model used for faceplate has a considerable impact on the load-carrying capacity of a sandwich panel with global imperfection.

- In the presence of local imperfections, material models used for both faceplates and core greatly affect the loading capacity of the sandwich panel. As per theory from Howard Allen, stress for face wrinkling can be calculated from the following formula,

$$\sigma_{cr} = B_1 E_f^{1/3} E_c^{2/3}$$

Where, B_1 is a constant base on poison's ratio of the core.

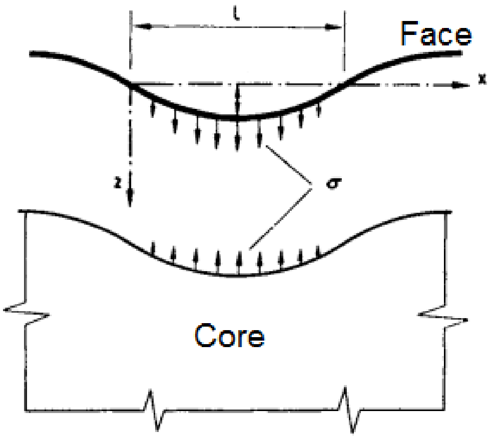


Figure 3.23 Stress between the faceplate and supporting elastic medium (core)

From this equation, it can be observed that critical stress is largely dependent on the modulus of elasticity of core. This can explain why the material model used for core has a considerable impact on the load-carrying capacity of a sandwich panel with local imperfection.

- In column buckling theory, an increase in the thickness of both faceplates and core has a positive impact on buckling resistance of the sandwich panel. Effect of an increase in core thickness is smaller than the thickness of faceplates.

4 Comparison between Stiffened Plates and Sandwich Panel

In this chapter, plate buckling theory will be applied to the sandwich panel and stiffened plate to calculate buckling resistance. Huisman structure is built with a stiffened plates. An arbitrary stiffened plate from this structure will be considered. One of the important load in Huisman structure is compressive load. So buckling resistance of stiffened plate will be calculated. Also, the weight of stiffened plate will be calculated. Afterwards, a sandwich panel with proper configuration will be chosen which has the same buckling resistance as that of a stiffened plate under the same boundary condition. Three different sandwich panels will be considered, the sandwich panel with faceplates S355, S690 & S1100. Base on steel grades proper sandwich configuration will be chosen which will give same buckling resistance as that of the stiffened plate but reduced self-weight. Results of all sandwich panels will be compared with the stiffened plate to determine maximum weight reduction that can be achieved with a sandwich panel with faceplates of specific steel grade.

4.1 Assumptions

- The load is applied only on faceplates, not on the core
- Core does not contribute to buckling capacity (load carrying capacity) of sandwich panel
- The core has isotropic behaviour
- The density of core is same everywhere
- During the manufacturing process, the proper metallurgical bond is established between core and faceplates

4.2 Application of Plate Buckling Theory

4.2.1 Stiffened Plate

Effective Width Method:-

Effective width method can be used if the following requirements are satisfied,

- Sub-panels of plate and plate itself is rectangular or nearly rectangular with flanges deviating from horizontal less than 10 degrees.
- Panels can be stiffened or unstiffened. Stiffeners can be in a longitudinal direction or transverse direction or in both directions.
- Cutouts and unstiffened openings should be small, with diameters less than 0.05 b, where b is the width of the plate element. Properly stiffened holes may be larger, but EN 1993-1-5 does not provide any design rules
- Cross-section of the member should be uniform. If the thickness of the panel is not constant, then the equivalent thickness should be taken equal to the smallest one.
- Flange induce buckling should be prevented by selecting appropriate web slenderness.

Principle of effective width method:-

In order to determine the resistance of cross-section class 4 subjected to direct stress by effective width method, effective widths of each plate element in compression are calculated independently. Afterwards, effective cross-section area A_{eff}, an effective moment of inertia I_{eff} and effective section modulus W_{eff} can be calculated based on these calculated effective widths. If, shear lag is relevant effects must be included. For elements under compression, the combined effect of shear lag and plate buckling should be taken into account for the calculation of effective widths.
If axial force and bending act simultaneously, the calculation of effective width can be based on resulting stress distribution. Euro code 1993-1-5 give simplified way for calculations.

Plate-like buckling of longitudinally stiffened plate results in global buckling of panel i.e. buckling of the plate as well as stiffeners. If sub-panels are slender then they may buckle. In such cases interaction of local and global plate buckling should be considered.

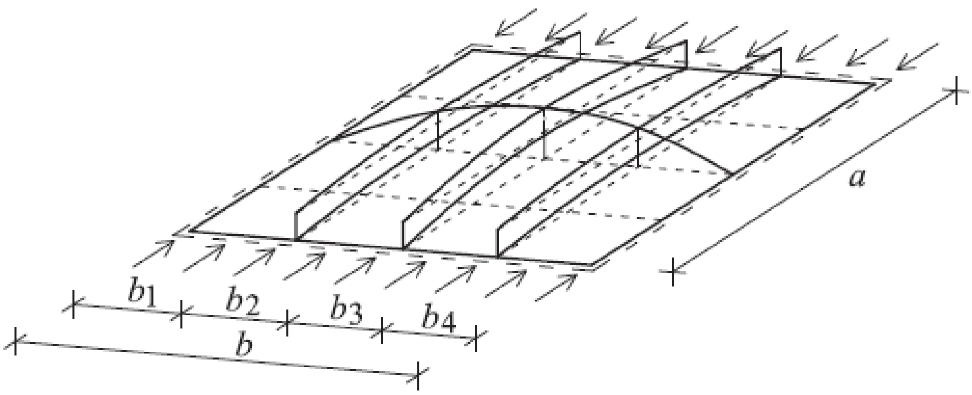


Figure 4.1 Plate like buckling of a stiffened plate [11]

This local and global interaction can be taken into account by modifying plate slenderness

$$\lambda_p = \sqrt{\frac{N_y}{N_{cr}}} = \sqrt{\frac{\beta_{A,C} f_y}{\sigma_{cr,p}}} \tag{4.1}$$

where,

$\sigma_{cr,p}$ is elastic critical buckling stress of a stiffened plate

$$\beta_{A,C} = \frac{A_{C,eff,loc}}{A_C} \tag{4.2}$$

$A_{C,eff,loc}$ is a sum of effective areas of sub-panels and stiffeners according to sub-section 2.4.3.1, excluding edge parts along longitudinal edges. Sub-panels are assumed to be fully supported by stiffeners (no global buckling of stiffeners), see Figure 4.1.

$$A_{C,eff,loc} = A_{sl,eff} + \sum_i \rho_{loc,i} b_{loc,i} t \tag{4.3}$$

$A_{sl,eff}$ is the sum of effective areas of longitudinal stiffeners

$b_{loc,i}$ is the width of each individual sub-panel *i*

$\rho_{loc,i}$ is reduction factor of each sub-panel *i*

A_C is a gross cross-section of compression zone of stiffened plate excluding edge parts along longitudinal edges

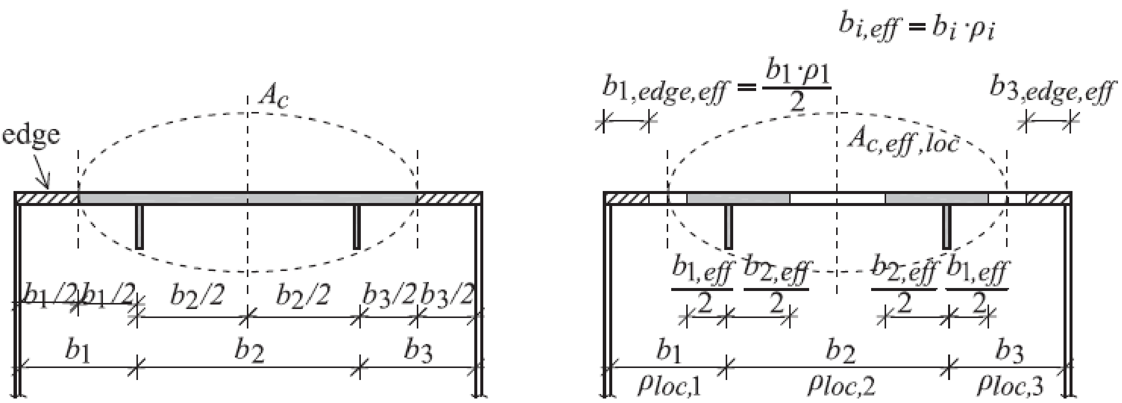


Figure 4.2 Stiffened plate under uniform compression [11]

When the plate is under a stress gradient, effective width and contributing widths of gross area are determined according to tables given in euro code. For the web of a plate girder, principles for the determination of $A_{C,eff,loc}$ and A_C are shown in Figure 4.2.

For longitudinally stiffened plates, elastic critical stress $\sigma_{cr,p}$ can be calculated in several possible ways. Basic expression for $\sigma_{cr,p}$is same as for unstiffened plates. But the calculation of plate buckling coefficient $K_{\sigma,P}$ is more complex

$$\sigma_{cr,p} = K_{\sigma,p} \frac{\pi^2 E}{12(1-v^2)} \left(\frac{t}{b}\right)^2 \tag{4.4}$$

$$\sigma_{cr,p} = K_{\sigma,p}\sigma_E \tag{4.5}$$

Plate buckling coefficient $K_{\sigma,P}$ maybe determined by:

- Design charts for smeared or discretely spaced stiffeners
- Simplified analytical expressions (two such procedures are given in EN 1993-1-5)
- Computer simulations

A very simple method is the use of design charts. But its applicability is limited to a certain range of charts. In these charts for discretely spaced stiffener, the value of $k_{\sigma,p}$ is cut off at the onset of local buckling of sub-panels. Therefore the use of these charts is limited. Well known charts are Kloppel charts. Kloppel charts contain values of $k_{\sigma,p}$ for smeared stiffeners as well as for discretely spaced stiffeners.

Three or more equally spaced stiffeners with aspect ratio a/b >= 0.5

The whole plate must be in compression with edge stress ratio $\Psi = {}^{\sigma_1}/_{\sigma_2} \geq 0.5$

if $\alpha \leq \sqrt[4]{\gamma}$

$$k_{\sigma,p} = \frac{2((1+\alpha^2)^2 \pm 1)}{\alpha^2(\Psi+1)(1+\delta)} \tag{4.6}$$

if $\alpha > \sqrt[4]{\gamma}$

$$k_{\sigma,p} = \frac{4(1+\sqrt{\gamma})}{(\Psi+1)(1+\delta)} \tag{4.7}$$

where,

$$\gamma = {}^{I_{sl}}/_{I_p} \tag{4.8}$$

$$\delta = {}^{A_{sl}}/_{A_p} \tag{4.9}$$

$$\alpha = {}^{a}/_{b} \geq 0.5 \tag{4.10}$$

$$I_p = \frac{bt^3}{12(1-\upsilon^2)} \tag{4.11}$$

I_{sl} is the second moment of area of whole stiffened plate

I_p is the second moment of area of the plate itself

A_{sl} is the sum of the gross area of individual longitudinal stiffeners

A_p is gross cross-section area of the plate

σ_1 is larger edge stress

σ_2 is smaller edge stress

According to winter formula reduction factor can be calculated as follows,

$$\rho = \frac{1}{\lambda_p}\left(1 - 0.22\frac{1}{\lambda_p}\right) \tag{4.12}$$

Or as per euro code, 1993-1-5 following formula can be used,

$$\rho = \frac{\lambda_p - 0.055(3+\Psi)}{\lambda_p^2} \leq 1 \tag{4.13}$$

Buckling resistance can be given by,

$$N_{bRd} = \rho \, A_C \, f_y \tag{4.14}$$

Weight: - the weight of stiffened plate per unit length 'mm' can be calculated as follows,

$$W = A * \rho \tag{4.15}$$

$$A = bt + nh_s t_s \tag{4.16}$$

where,

A is cross-section area of stiffened plate

ρ is the density of material (steel)

b is the width of the base plate

t is the thickness of the base plate

h_s is height/depth of stiffener

t_s is the thickness of the stiffener

n is the number of stiffeners

Please refer to Appendix B and Appendix C for calculation of buckling resistance and weight of considered stiffened plate.

4.2.2 Sandwich Panel

A sandwich panel under compression can have different modes of failure. The sandwich panel can fail due to buckling, wrinkling of sandwich faceplates, crushing of core, core under compression called as core crimpling. When the sandwich is subjected to in-plane compressive load, it could fail by buckling of complete panel i.e. global buckling.

Consider the sandwich panel with length 'a' and width 'b'. The sandwich is made of steel plates (flange) separated by aluminium foam. The thickness of the sandwich panel is a sum of the thickness of faceplates 't_f' and thickness of core 't_c'. Both faceplates are identical i.e. of the same thickness and same material. In addition, as said previously core has isotropic behaviour.

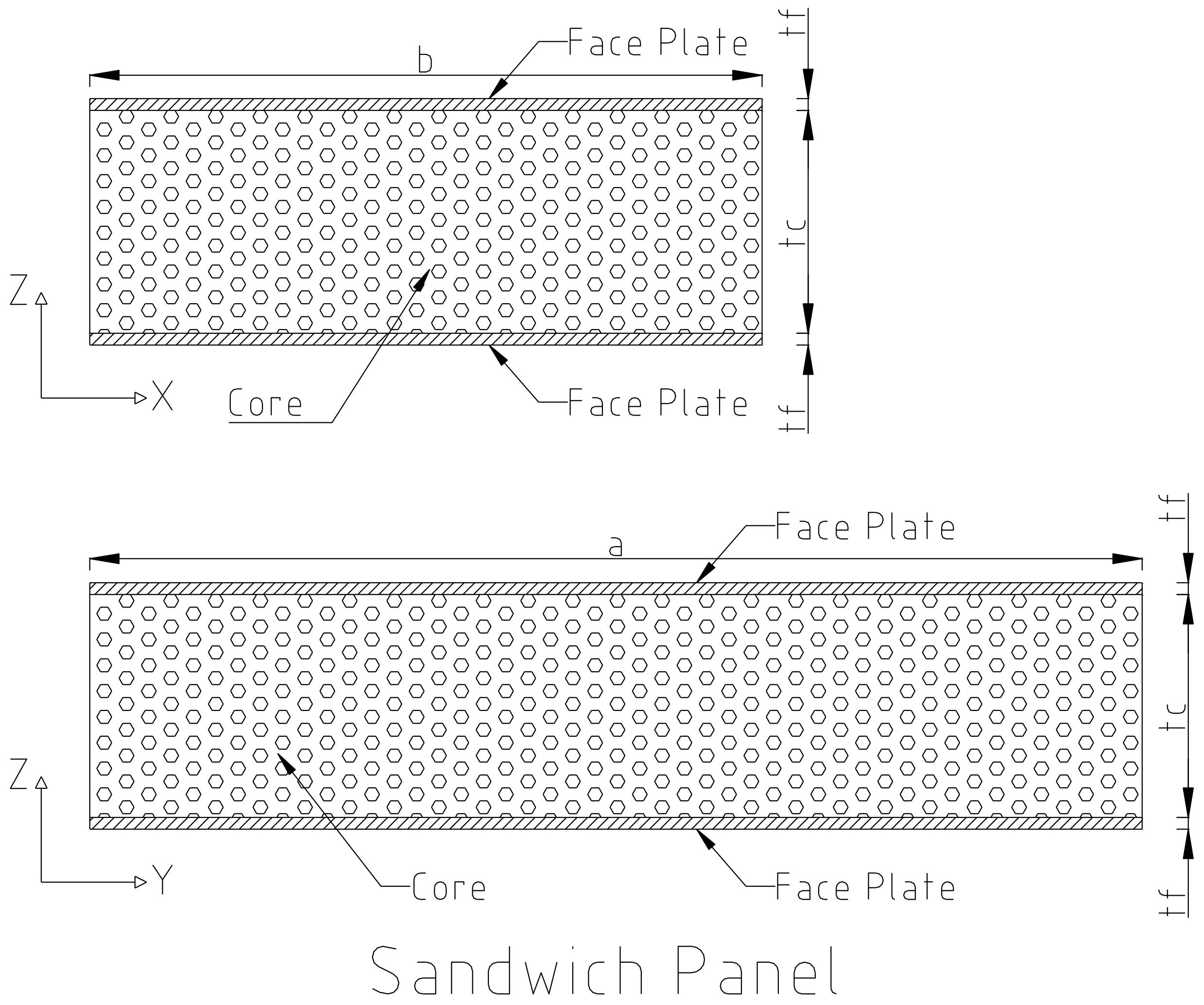


Figure 4.3 General configuration of the sandwich panel

In the above figure,

$a = lenght$

$b = widht$

$t_f = thickness\ of\ face\ plate$

$t_c = thickness\ of\ core$

Global Buckling:-

When the sandwich panel is subjected to compressive load, failure can occur due to buckling of the complete panel also called as global buckling. In order to understand the buckling behaviour of sandwich panel, analysis is performed on various sandwich panels. Jack Vinson described critical global buckling load for the sandwich panel under unidirectional in-plane compressive loading in his book. In equation, D_1, D_2, D_3 are values of flexural stiffness, 'a' is length & 'b' is the width of sandwich, 'm' and 'n' represents the number of half-sines in the longitudinal and transverse direction respectively. N_{cr} gives critical buckling load per unit width as follows,

$$N_{cr} = -\frac{\pi^2 a^2}{m^2}\left[D_1\left(\frac{m}{a}\right)^4 + 2D_3\left(\frac{m}{a}\right)^2\left(\frac{n}{b}\right)^2 + D_2\left(\frac{n}{b}\right)^4\right] \tag{4.17}$$

Where,

$$D_1 = D_2 = D_3 = \frac{E_f t_c^2 t_f}{2(1-\upsilon_f^2)} \tag{4.18}$$

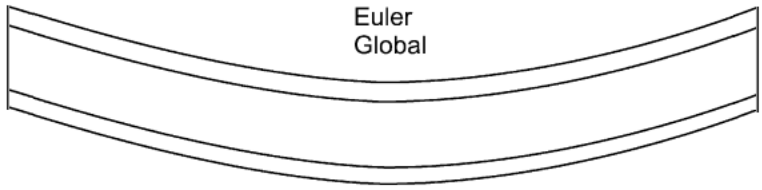


Figure 4.4 Global buckling of the sandwich panel under compressive loading [8]

The lowest value of 'N_{cr}' is important. All values of 'n' appear in the numerator so in order to get the lowest value of 'N_{cr}' value of 'n' should be equal to 1. Number of half sines in the longitudinal direction is given by 'm'. In the equation of 'N_{cr}', 'm' appears in several places, and depending on aspect ratio & flexural stiffness it is not clear which value of 'm' gives the lowest value of 'N_{cr}'. However, for the given plate and condition, it can be easily determined computationally.

From this value of 'N_{cr}' critical stress can be determined. Critical stress can be used to determine the slenderness of the plate. By referring to winter formula, for calculated slenderness corresponding reduction factor can be derived which will be used to determine designed strength of plane. Finally buckling strength of the panel is the product of reduction factor, yield strength and cross-section of both faceplates.

Face Stress: -

Assume that core does not contribute to buckling capacity of sandwich panel i.e. in-plane load are resisted only by faceplates, not by the core. Therefore compressive stresses in loading direction will be,

$$\sigma_x = N_x/2t_f \tag{4.19}$$

Where, N_x is load per unit width, t_f is the thickness of plates. Face stress σ_x will be restricted to yield strength of plates so that yielding of plates will be avoided.

Core shear instability:-

According to Jack Vinson, core shear instability or crimping will occur at face stress lower than that of overall panel buckling when,

$$V_x \geq k_1 B_1 r \tag{4.20}$$

In the case of core shear instability, critical stress can be given by,

$$\sigma_{cr} = \frac{G_c t_c}{2t_f} \tag{4.21}$$

Overall panel buckling and core shear instability will occur at same face stress value when $V_x = k_1 B_1 r$ or more specifically when $\frac{\pi^2}{2(1-\nu_{xy}\nu_{yx})}\frac{\overline{E_{fx}}}{G_c}\frac{t_c t_f}{b^2} = k_1 r$

$$\frac{\pi^2}{2(1-\nu_{xy}\nu_{yx})}\frac{\overline{E_{fx}}}{G_c}\frac{t_c t_f}{b^2} = k_1 r \tag{4.22}$$

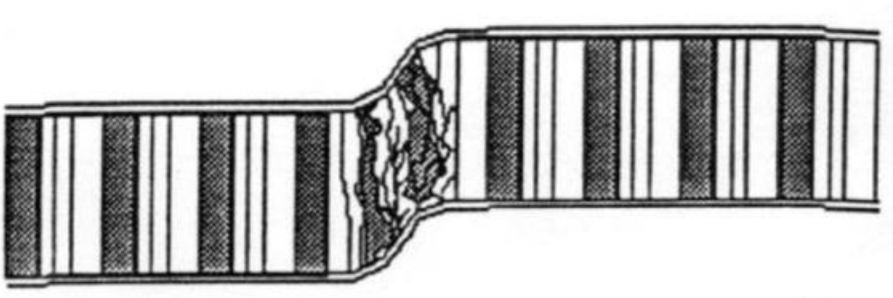


Figure 4.5 Core shear instability [3]

Face wrinkling: -

Local failure can also occur in the sandwich panel. Faceplates of the sandwich panel can fail locally. This failure mechanism is called as wrinkling. In this type of failure, buckling of a small portion of faceplate occurs. In this failure, faceplates of sandwich panel buckles into or out of core i.e. out of plane buckling of faceplates happens. In the case of thin plates, short-wavelength buckling can occur in addition to panel buckling and core shear instability. This face wrinkling can be described by,

$$\sigma_{cr} = \left[\frac{2t_f E_c \left(E_{fx}E_{fy}\right)^{\frac{1}{2}}}{3t_c\left(1-\nu_{xy}\nu_{yx}\right)}\right]^{1/2} \tag{4.23}$$

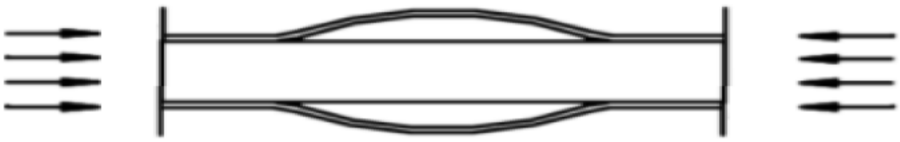


Figure 4.6 Face wrinkling of a sandwich panel under compressive loading [15]

Weight: -

Weight of sandwich panel per unit length can be calculated as follows,

$$W = (2t_f\rho_f + t_c\rho_c)b \tag{4.24}$$

where,

ρ_f is the density of flanges

ρ_c is the density of the core

G_c is the shear modulus of the core along the x-direction

ν_{xy} & ν_{xy} is Poisson's ratio

E_{fx} & E_{fy} is the modulus of elasticity of faceplates in x and y-direction

E_c is the modulus of elasticity of the core

Please refer to Appendix D and Appendix E for sample calculation of buckling resistance and weight of the sandwich panel.

4.3 Comparison

In order to replace this stiffened plate by the sandwich panel, the same buckling analysis should be done on the sandwich panel. For a proper and fair comparison, consider a sandwich panel having the same length and width as of stiffened plate. Yield strength of faceplates and the stiffened plate is same. In an analysis, the same boundary conditions are applied to the stiffened plate and sandwich panel. The variable in this case of the sandwich panel is its thickness, which is composed of the thickness of faceplates and core. This thickness will be varied so as to achieve the same buckling strength as that of the stiffened plate along with weight reduction.

Considering stiffened plate used in Huisman structure and sandwich panel with arbitrary configuration but having the same dimensions as that of considered stiffened plate. Buckling resistance and weight of both stiffened plate and sandwich panel is calculated as per theory introduced in chapter 4.2. For sample calculation of buckling resistance of stiffened plate and sandwich panel, refer to Appendix B and Appendix D respectively. The following figures will illustrate considered sandwich panels and stiffened plate.

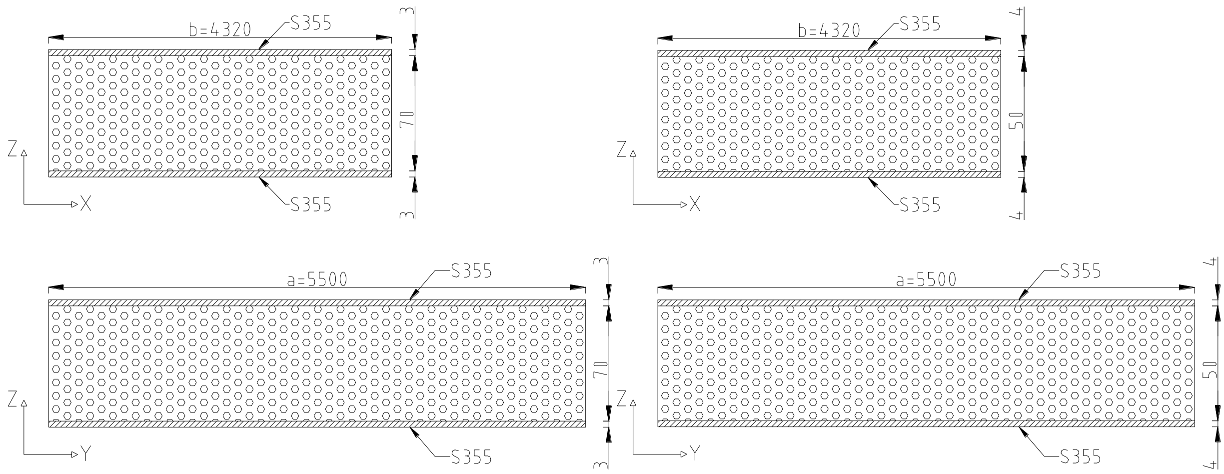


Figure 4.7 Sandwich Panel with arbitrary configuration 3-70-3 and 4-50-4

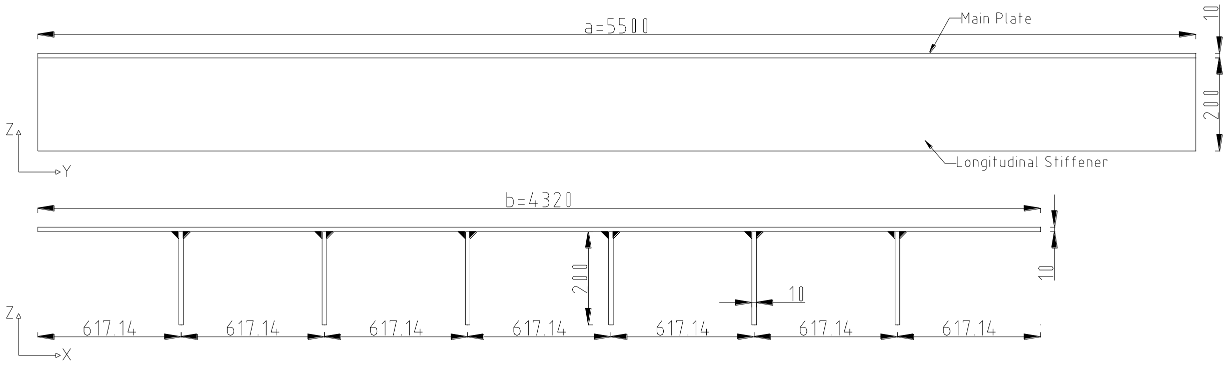


Figure 4.8 Stiffened Plate

4.3.1 Buckling Resistance

The sandwich panel is new material & its behavior is not known. It is not clear how the sandwich panel will fail. Therefore, it is important to know or evaluate different failure modes & stresses corresponding to it. From this, it can be shown that the global buckling of the sandwich panel is critical than any other failure mode. This is because critical stress calculated for global buckling is less than critical stress for other failure modes. Stiffened plate which is chosen for analysis is designed by Huisman. From their design, we know that global buckling of plate is critical.

Various properties of sandwich panel are calculated and they are arranged in the following table. For sample calculation of buckling resistance of stiffened plate refer to Appendix B For sample calculation of buckling resistance of sandwich panel refer to Appendix D.

Properties	*Sandwich 4-50-4*	*Sandwich 3-70-3*
Critical Buckling Stress (MPa)	320	630
Buckling resistance (KN)	9200	8700
Critical stress Core Shear (MPa)	1200	2200
Critical stress Face Wrinkling (MPa)	2500	1800

Table 4.1 Comparison of various calculated parameters

Buckling resistance of stiffened plate and sandwich panel is calculated and compared in the following table,

Fields of Comparison	*Stiffened Plate*	*Sandwich 4-50-4*	*Sandwich 3-70-3*
Critical Buckling Stress (MPa)	180	320	630
Buckling resistance (KN)	12300	9200	8700

Table 4.2 Comparison of between sandwich panel & stiffened plate

4.3.2 Weight

Weight of stiffened plate and sandwich panel is calculated and is shown in the following table. For sample calculation, please refer to Appendix C and Appendix E.

Fields of Comparison	*Stiffened Plate*	*Sandwich 4-50-4*	*Sandwich 3-70-3*
Weight per unit length (gram/mm)	433	423	415
Total weight (KG)	2380	2330	2280
Weight reduction per unit length (gram/mm)	-	10	18
Weight reduction for panel (KG)	-	60	100
Percent weight reduction	-	2.3	4.2

Table 4.3 Comparison between the sandwich panel and stiffened plate

From the above tables, we can observe that considered sandwich panel has self-weight lower than that of the stiffened plate but does not gives required buckling resistance.

Aim of the study is to design a sandwich panel, which will be appropriate to replace this stiffened plate & results in weight reduction. So in order to achieve both criteria, it would be better to plot the relationship between the thickness of core and thickness of faceplates of the sandwich to achieve the same buckling resistance as that of the stiffened plate. Similarly, the same relationship can be plotted to fulfil weight requirement. By equating buckling resistance of sandwich panel and stiffened plate, the relation between the thickness of the core and thickness of faceplate can be found out and plotted. These graphs will give an idea about how the thickness of the core will vary if the thickness of faceplates is varied or vice versa to achieve desired buckling resistance. The graph is shown below,

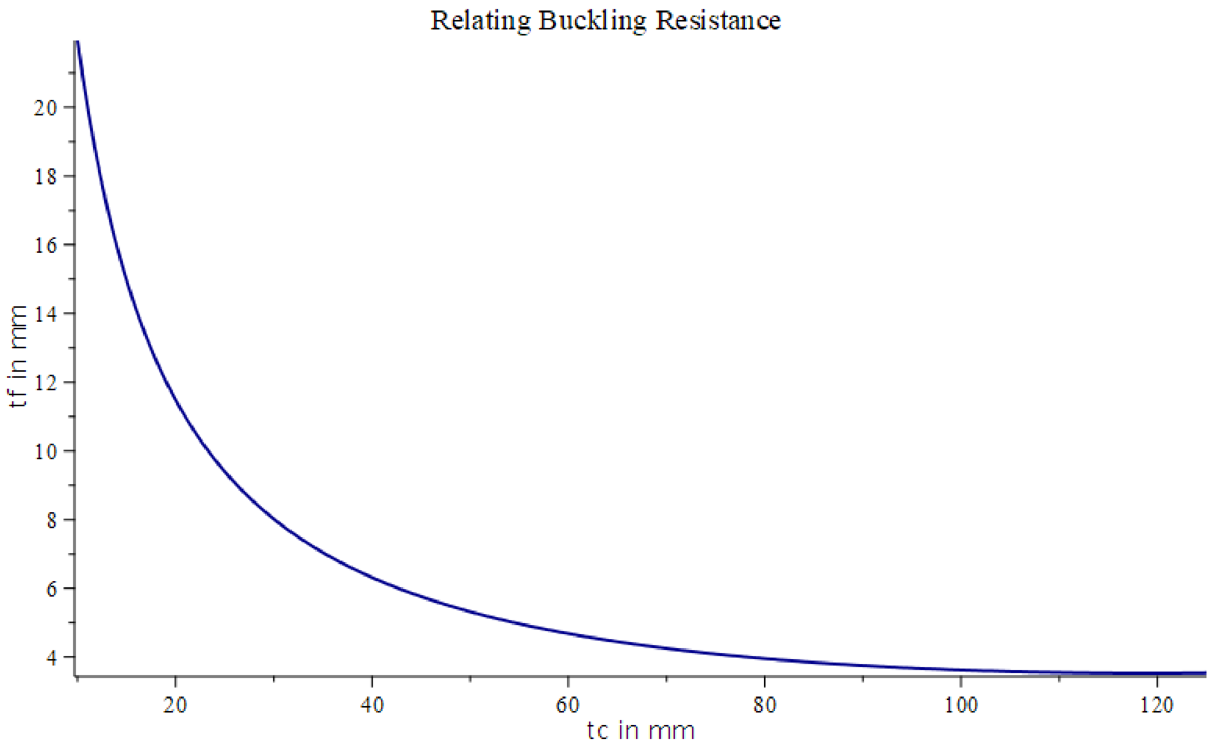


Figure 4.9 Relation between the thickness of the core and faceplate to achieve the same buckling resistance of the stiffened plate

Similarly, the graph can be plotted to give an idea about, how the thickness of core will vary if the thickness of faceplates is varied or vice versa so that, the weight of stiffened plate and sandwich panel will be same.

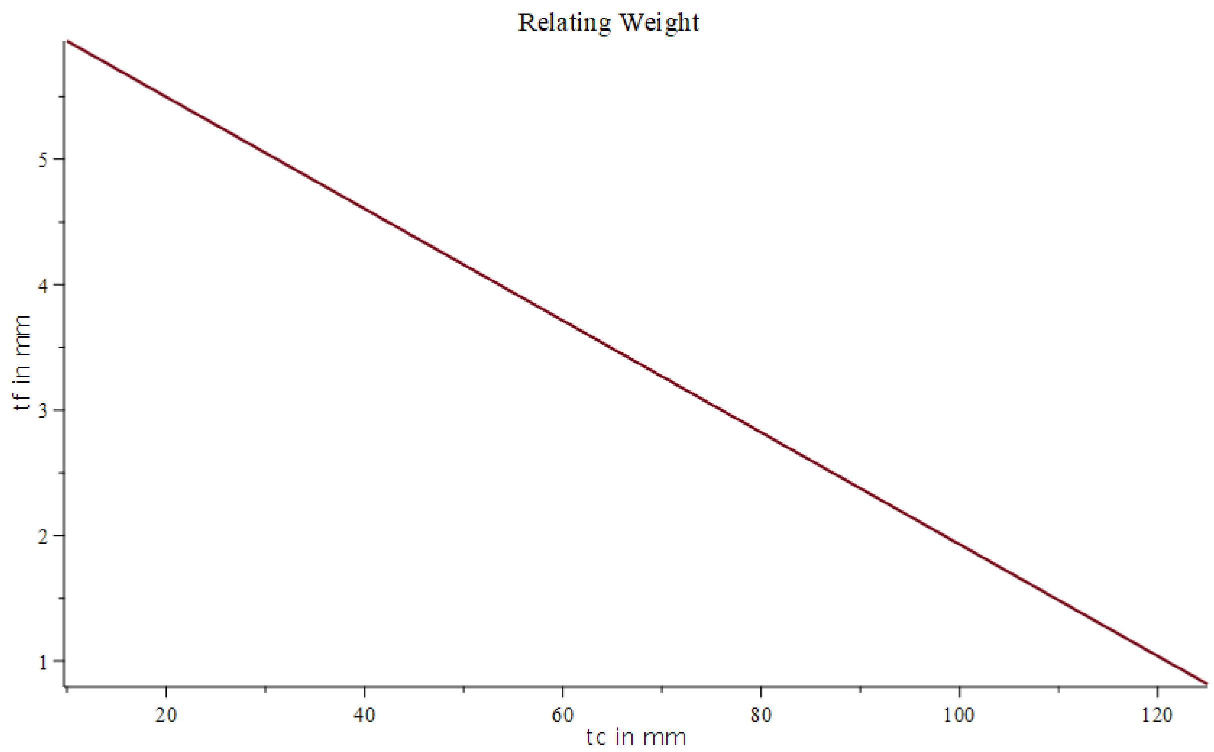


Figure 4.10 Relation between the thickness of the core and faceplate to achieve the same weight as that of stiffened plate

From these two graphs, it will be easy to select a sandwich panel with proper configuration which will fulfil both criteria of same buckling resistance and reduced weight. This will also reduce the number of trial and error required for the selection of sandwich with proper configuration. This concept of graphs will be used in the coming chapters.

4.4 Replacing Stiffened Plate by Sandwich Panel

There can be a lot of configurations of a sandwich panel which can be considered and evaluated to see if same buckling resistance and weight reduction is achieved. But this is time consuming and long process. Therefore the relationship between the thickness of core and faceplate can be found out, to achieve the same buckling resistance and reduced weight, as stated in the previous chapter. From this relation, the desired sandwich configuration can be found out and used for replacement of stiffened plate. If both graphs are combined, the range of thickness of core and thickness of faceplates can found out with which both objectives reduced weight and same resistance are achieved. In this chapter, three different sandwich panels will be considered with different yield namely S355, S690 and S1100. Buckling resistance and weight of these panels will be calculated and compared with stiffened plate, to evaluate if the selected sandwich gives the same buckling resistance and weight reduction at the same time.

4.4.1 Sandwich with Faceplates S355

Sandwich 5.4-50-5.4

Consider sandwich with faceplates S355. Two graphs are plotted to see the relationship between the faceplate and core thickness as stated before. These graphs are combined together as shown in Figure 4.11. In graph brown line represent the relation between the thickness of core and faceplate to achieve the same weight whereas the blue line represents the relation between the thickness of core and faceplate for buckling resistance.

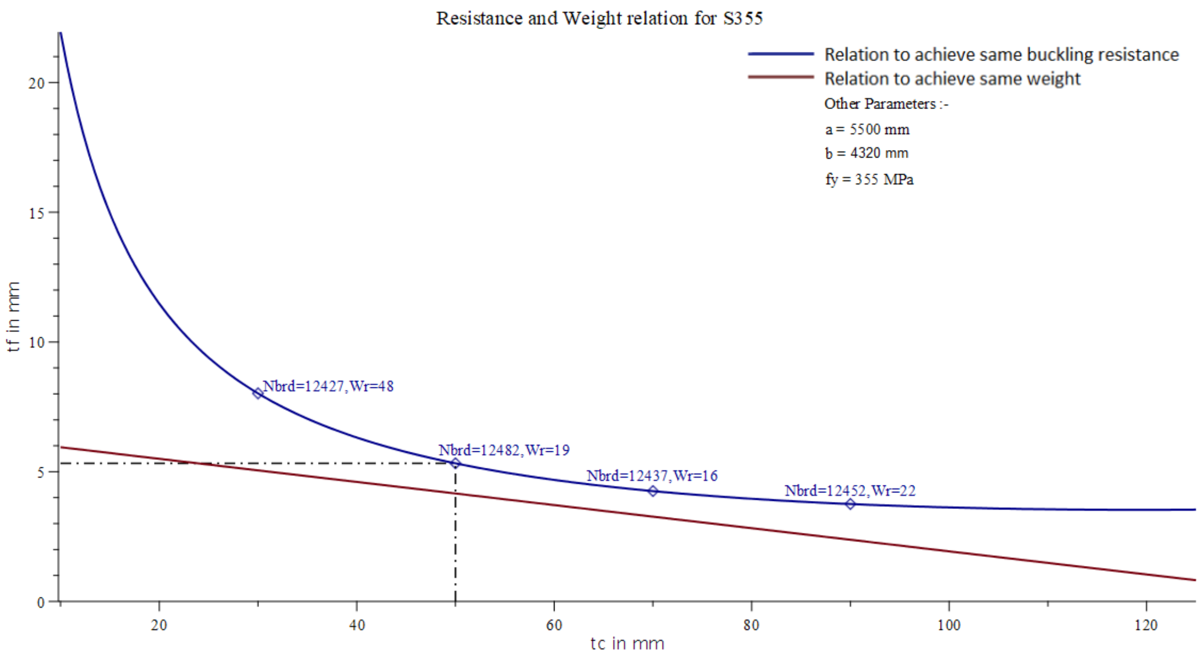


Figure 4.11 Relation between the thickness of faceplate and core for buckling resistance and weight

From the above graph, it can be observed that a sandwich with faceplates S355 cannot fulfil both criteria simultaneously. Configuration of the sandwich panel to achieve the same buckling strength as that of the stiffened plate will have more weight than that of the stiffened plate. For example consider sandwich 5.4-50-5.4, buckling resistance of the sandwich panel is 12482 KN and weight 2844 KG. It has 19% more weight than the stiffened plate. Therefore, sandwich 5.4-50-5.4 cannot be used to replace the stiffened plate to achieve weight reduction.

Considered a sandwich with a core thickness of 50 mm and faceplate thickness of 5.4 mm is shown in the following Figure 4.12.

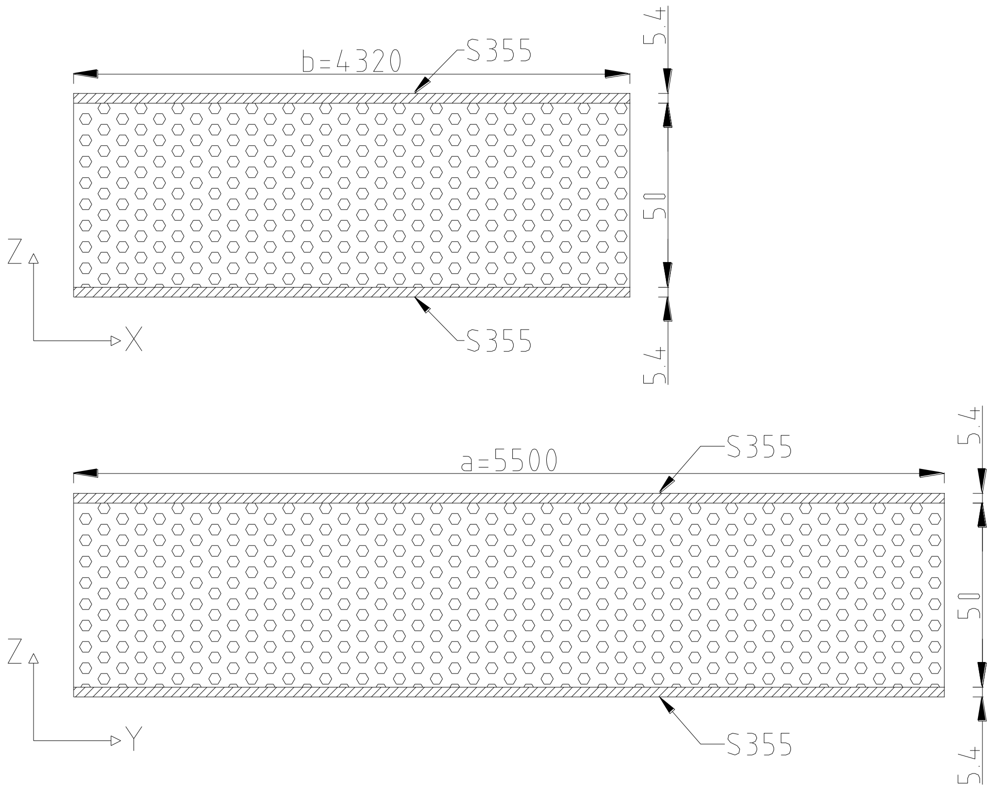


Figure 4.12 Sandwich Panel 5.4-50-5.4

Various properties of this sandwich are calculated and arranged in the following table.

Properties	*Sandwich 5.4-50-5.4*
Critical Buckling Stress (MPa)	320
Buckling resistance (KN)	12500
Critical stress Core Shear (MPa)	890
Critical stress Face Wrinkling (MPa)	2900

Table 4.4 Properties of sandwich 3.6-50-3.6

Fields of Comparison	*Stiffened Plate*	*Sandwich 5.4-50-5.4*
Critical Buckling Stress (MPa)	180	320
Buckling resistance (KN)	12300	12500

Table 4.5 Comparison of between sandwich panel & stiffened plate

Fields of Comparison	*Stiffened Plate*	*Sandwich 5.4-50-5.4*
Weight per unit length (gram/mm)	430	520
Total weight (KG)	2380	2840
Weight reduction per unit length (gram/mm)	-	+80
Weight reduction for panel (KG)	-	+460
Percent weight reduction	-	+19

Table 4.6 Comparison between the sandwich panel and stiffened plate

If sandwich 5.4-50-5.4 with faceplates of grade S355 is used then we can achieve buckling strength more than that of the stiffened plate. However, with the sandwich configuration that self-weight of the structure is increased. Increase in self-weight is 460 KG as compared to a stiffened plate which is 19 % of the stiffened plate. So the criterion of weight reduction is not achieved.

4.4.2 Sandwich with Faceplates S690

Sandwich 3.6-50-3.6

Consider sandwich with faceplates S690. Two graphs are plotted to see the relation between the faceplate thickness and core thickness as stated before. These graphs are combined together as shown in Figure 4.13. In graph brown line represent the relation between the thickness of core and faceplate to achieve the same weight whereas the green line represents the relation between the thickness of core and faceplate for buckling resistance.

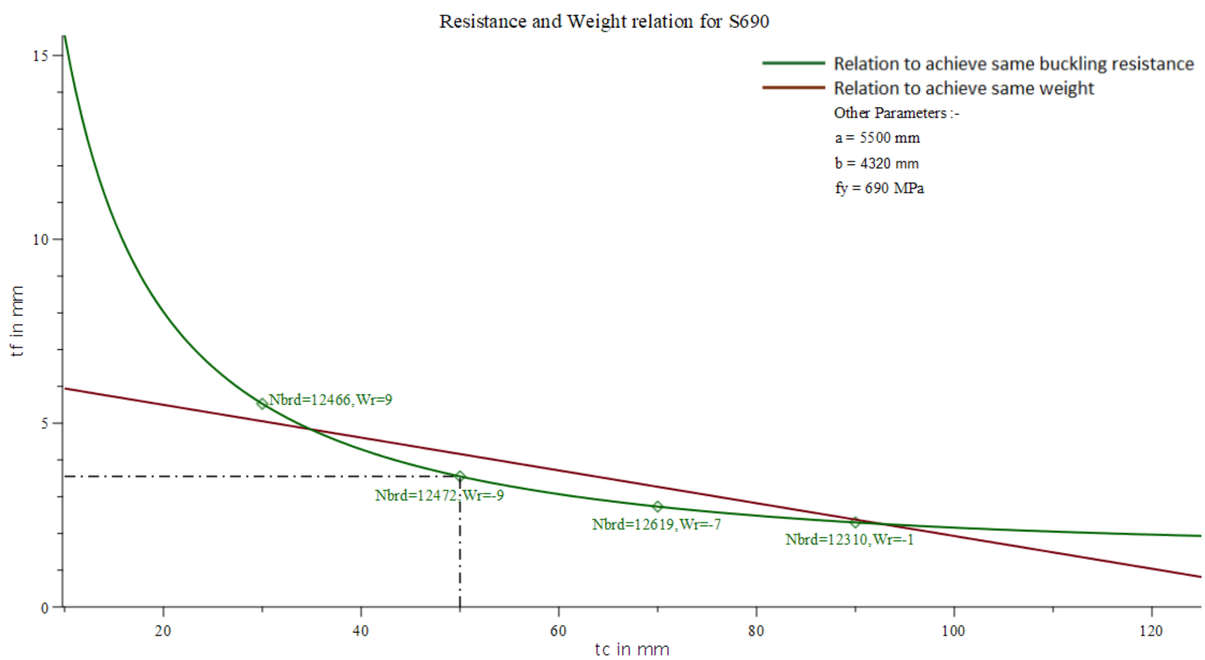


Figure 4.13 Relation between the thickness of faceplate and core for buckling resistance and weight

From the above graph, we can predict a range of thickness of core and faceplate to achieve the same buckling resistance and lower weight as that of the stiffened plate. In the case where steel grade S690

is used for faceplates of sandwich panel, a range for the thickness of faceplate is between 2.1 mm to 5 mm and corresponding range for core thickness is between 90 mm to 35 mm approximately.
Consider a sandwich with a core thickness of 50 mm and faceplate thickness of 3.6 mm as shown in the following Figure 4.14.

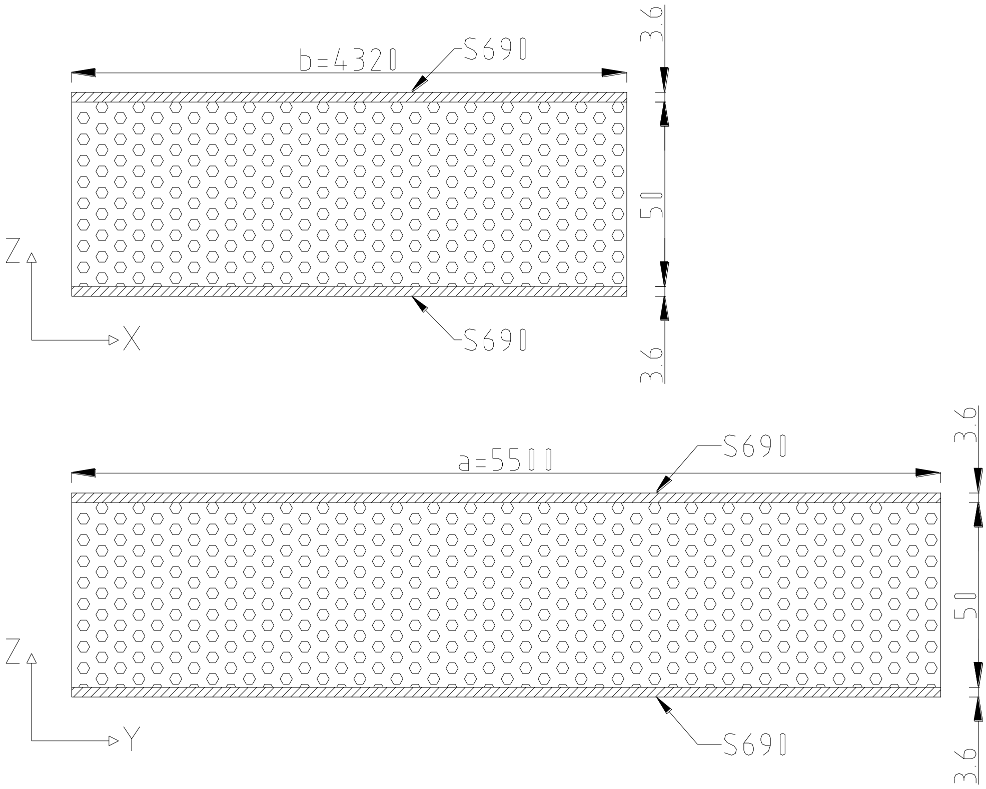


Figure 4.14 Sandwich Panel 3.6-50-3.6

Various properties of this sandwich are calculated and arranged in the following table. For calculation of buckling resistance of sandwich 3.6-50-3.6 refer to Appendix D.1 & Appendix D.2

Properties	*Sandwich 3.6-50-3.6*
Critical Buckling Stress (MPa)	320
Buckling resistance (KN)	12500
Critical stress Core Shear (MPa)	1300
Critical stress Face Wrinkling (MPa)	2400

Table 4.7 Properties of sandwich 2.4-50-2.4

Fields of Comparison	*Stiffened Plate*	*Sandwich 3.6-50-3.6*
Critical Buckling Stress (MPa)	180	320
Buckling resistance (KN)	12300	12500

Table 4.8 Comparison of between sandwich panel & stiffened plate

Fields of Comparison	*Stiffened Plate*	*Sandwich 3.6-50-3.6*
Weight per unit length (gram/mm)	430	400
Total weight (KG)	2380	2170
Weight reduction per unit length (gram/mm)	-	-40
Weight reduction for panel (KG)	-	-210
Percent weight reduction	-	-9

Table 4.9 Comparison between the sandwich panel and stiffened plate

If sandwich 3.6-50-3.6 with faceplates of grade S690 is used then buckling strength more than that of the stiffened plate can be achieved. Also, with this sandwich configuration, it can be observed that self-weight of the structure is reduced. Reduction of self-weight is 210 kg as compared to the stiffened plate which is 9 % of the stiffened plate.

4.4.3 Sandwich with Faceplates S1100

Sandwich 2.8-50-2.8

Consider sandwich with faceplates S1100. Two graphs are plotted to see the relation between faceplate thickness and core thickness as stated before. These graphs are combined together as shown in Figure 4.15. In graph brown line represent the relation between the thickness of core and faceplate to achieve the same weight whereas the violet line represents the relation between the thickness of core and faceplate for buckling resistance.

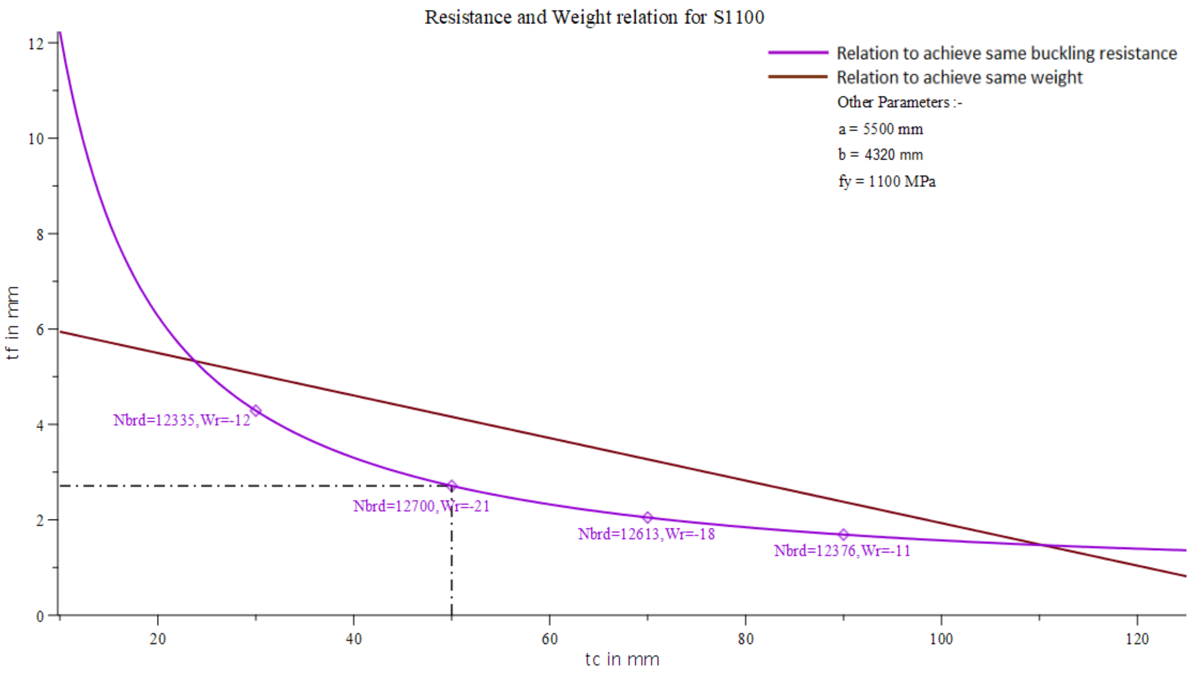


Figure 4.15 Relation between the thickness of faceplate and core for buckling resistance and weight

From the above graph, we can predict a range of thickness of core and faceplate to achieve the same buckling resistance and lower weight as that of the stiffened plate. In the case where steel grade S1100

is used for faceplates of sandwich panel, a range for the thickness of faceplate is between 1.5 mm to 5.2 mm and corresponding range for core thickness is between 110 mm to 25 mm approximately.

Consider a sandwich with a core thickness of 50 mm and faceplate thickness of 2.8 mm as shown in the following Figure 4.16.

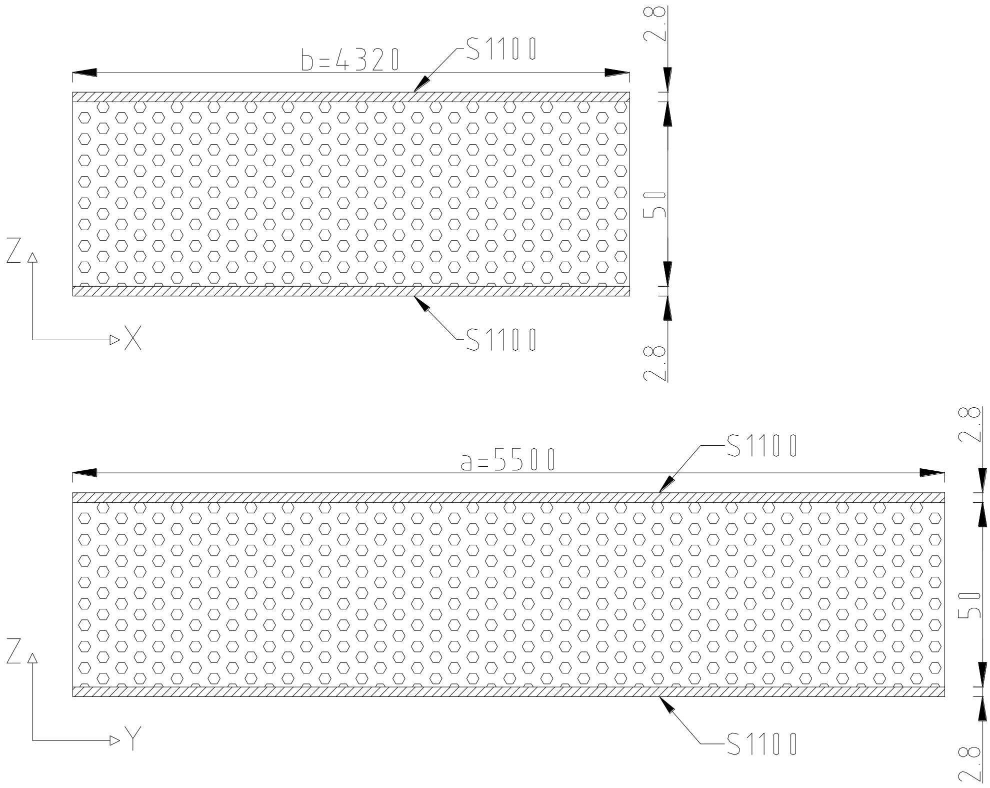


Figure 4.16 Sandwich Panel 2.8-50-2.8

Various properties of this sandwich are calculated and arranged in the following table. For calculation of buckling resistance of sandwich panel 2.8-50-2.8 refer to Appendix D.3 & Appendix D.4

Properties	*Sandwich 2.8-50-2.8*
Critical Buckling Stress (MPa)	320
Buckling resistance (KN)	12700
Critical stress Core Shear (MPa)	1700
Critical stress Face Wrinkling (MPa)	2100

Table 4.10 Properties of sandwich 2-50-2

Fields of Comparison	*Stiffened Plate*	*Sandwich 2.8-50-2.8*
Critical Buckling Stress (MPa)	180	320
Buckling resistance (KN)	12300	12700

Table 4.11 Comparison of between sandwich panel & stiffened plate

Fields of Comparison	*Stiffened Plate*	*Sandwich 2.8-50-2.8*
Weight per unit length (gram/mm)	430	340
Total weight (KG)	2380	1880
Weight reduction per unit length (gram/mm)	-	-90
Weight reduction for panel (KG)	-	-510
Percent weight reduction	-	-21

Table 4.12 Comparison between the sandwich panel and stiffened plate

If sandwich 2.8-50-2.8 with faceplates of grade S1100 is used then buckling strength of the sandwich panel is more than stiffened plate. Also, with this sandwich configuration, it can be observed that self-weight of the structure is reduced. Reduction of self-weight is 510 kg as compared to the stiffened plate which is 21 % of the stiffened plate.

4.5 Results and Observations

- Sandwich panel with high strength steel S355 can be used to replace stiffened plate from S355 steel but it cannot fulfil criteria of weight reduction.
- To achieve both criteria of weight reduction and the same buckling strength sandwich panel with extra high strength steel S690 or S1100 is required.
- Sandwich with faceplates S355 can be used to get the same buckling resistance as that of the stiffened plate from S355 but it results in an increase in self-weight by 19 %.
- Sandwich with faceplates S690 and S1100 fulfil both criteria of weight reduction and same buckling resistance.
- Sandwich panel with faceplates S690 results in a weight reduction of 9 % whereas sandwich panel with faceplates S1100 gives a weight reduction of 21 % as compared to S355 stiffened plate.

5 Optimization of Sandwich Panel

5.1 Effect of Parameters on Buckling Resistance

Similar to section 3.4, to understand the effect of various parameters on the buckling strength of the sandwich panel according to plate buckling, different graphs can be drawn. These graphs will explain how the buckling strength of the panel will change if one parameter changed and all other parameters are kept constant. There are three important parameters, which affects the buckling of sandwich panel. These parameters are the thickness of faceplate, the thickness of core and yield strength of faceplates.

For this task consider sandwich 4-50-4, having steel faceplates 4 mm thick and aluminium foam core 50 mm thick. Length and width of a sandwich is 5500 mm and 4320 mm respectively. Grade of steel is S355.

5.1.1 The Thickness of Faceplates

The faceplates have a major contribution in buckling strength of the sandwich panel. Its thickness is the most important parameter, which affects the buckling strength of the sandwich panel. Therefore, it is important to know the effect of thickness of faceplate on the buckling strength of the sandwich panel. Therefore plotting a graph between buckling resistance of the sandwich panel and thickness of faceplate while keeping all other parameters constant. The graph is illustrated as below.

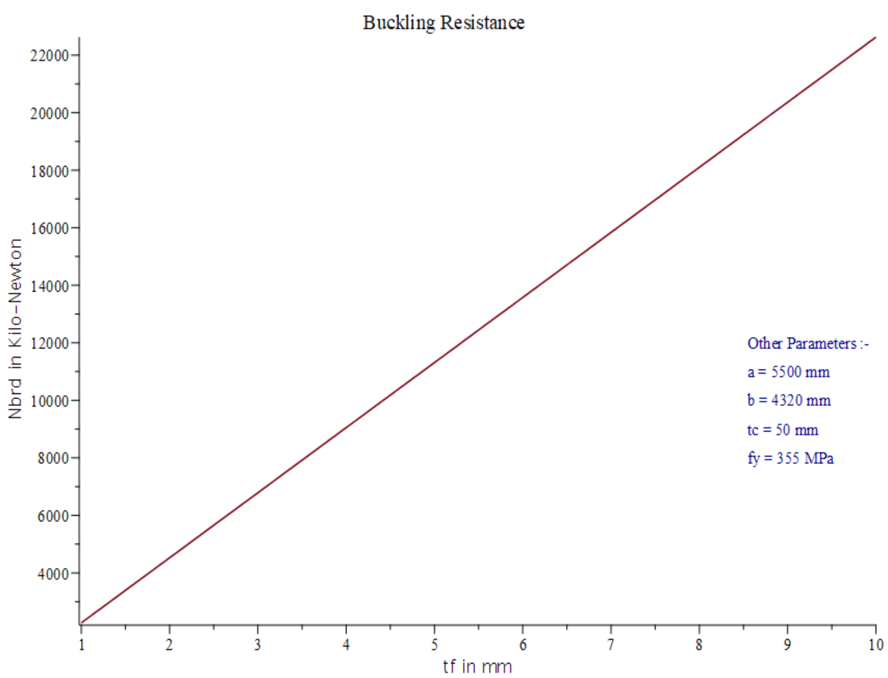


Figure 5.1 Effect of thickness of faceplate on the buckling load

From this graph, we can observe that there is a linear/direct relation between the thickness of faceplates and buckling strength of the sandwich panel.

5.1.2 Thickness of Core

The second parameter which affects the buckling of the sandwich is core thickness. The main function of the core is to maintain a specific distance between faceplates. Increase in core thickness will result in an increase in distance between outermost fibres of the faceplates and thus affecting buckling strength. Also, an increase in core thickness will increase the overall thickness of the sandwich panel and thus the moment of inertia is increased. This will result in a change in buckling resistance of the sandwich panel. Even though core itself has no significant contribution to buckling strength but because of above-explained effects, it is important to understand how the change in core thickness will affect the buckling strength of the sandwich panel.

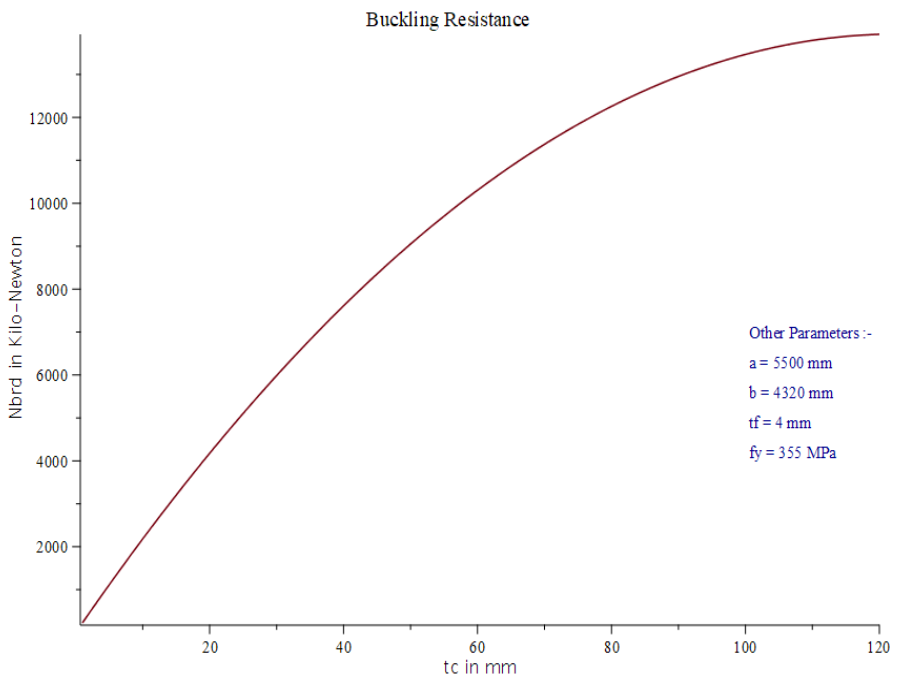


Figure 5.2 Effect of thickness of core on the buckling load

From the above graph, we can see that core thickness has a considerable effect on the buckling resistance of the sandwich panel. However, the effect of core thickness on bucking resistance is less than the effect of thickness of faceplates.

5.1.3 Yield Strength of Faceplates

The third parameter is the yield strength of faceplates. The faceplate can have any yield from 235 MPa to 1100 MPa. Allowable stress in faceplate will vary as per the grade of steel used. Buckling resistance of sandwich panel is the product of yield strength of faceplates, the cross-sectional area of faceplates and reduction factor. So buckling resistance of the sandwich panel is directly proportional to yield strength of faceplates

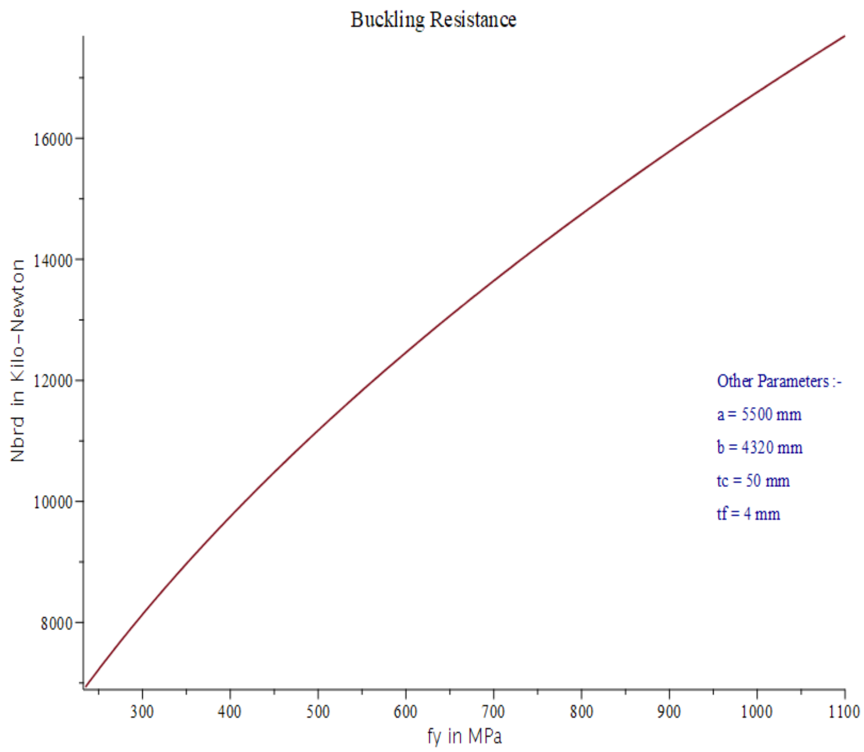


Figure 5.3 Effect of yield on the buckling load

Above graph shows that yield strength of faceplate has a significant impact on buckling resistance of the sandwich panel. Increase in yield strength will increase buckling resistance of the sandwich panel.

To understand the simultaneous effect of thickness of core and thickness of faceplates on buckling resistance of sandwich panel 3D graph can be plotted. In this graph, the thickness of core and faceplates are varied simultaneously and all the other parameters are kept constant. From the graph below, the configuration of the sandwich panel for a specific requirement can be determined approximately.

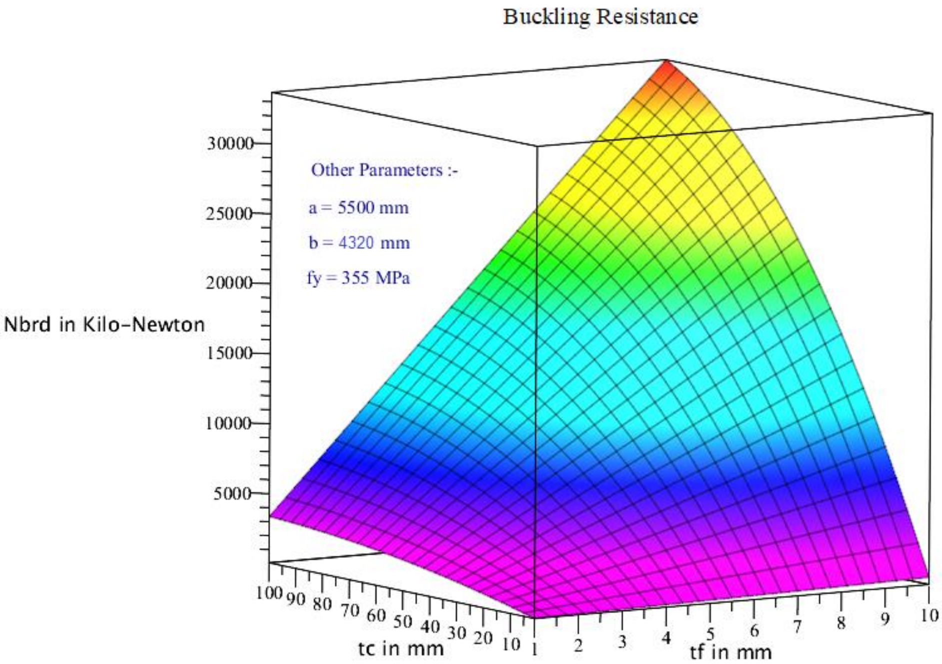


Figure 5.4 Effect of thickness of faceplate and core on buckling load simultaneously

Similarly, one more similar type of graph can be plotted which will illustrate the effect of thickness of faceplates and yield strength of faceplates on buckling resistance of the sandwich panel.

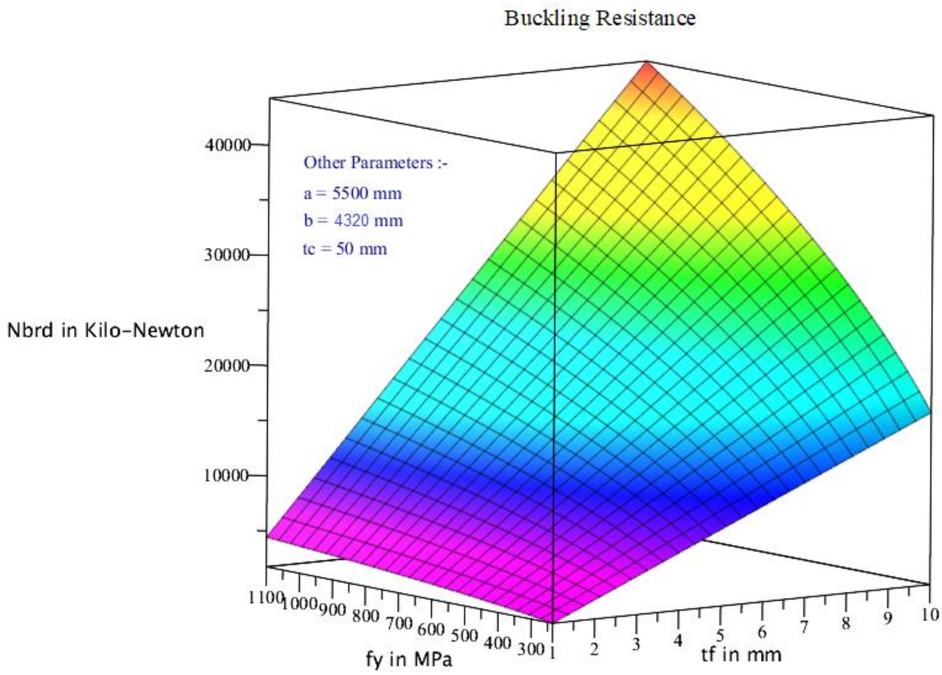


Figure 5.5 Effect of thickness and yield strength of faceplate on buckling load simultaneously

5.2 Optimisation of Sandwich Panel

Optimisation from two prospective are considered,

- Optimised sandwich for weight reduction
- Optimised sandwich in terms of use of yield strength of steel i.e. faceplate material utilisation for buckling

Optimisation of sandwich for weight reduction means the configuration of the sandwich panel, which will result in the highest weight reduction. In this context, there are only two parameters of sandwich panel namely thickness of core and faceplates which can be varied to achieve the highest weight reduction. For this purpose, it would be better if we can understand the effect of these two parameters on the buckling and weight of the sandwich panel. This can be observed and explained in section 5.1. With the main aim of weight reduction, a sandwich panel with proper combination faceplate and core thickness can be chosen. All combinations explained above (in section 4) are with this aim only.

Optimisation in other sense is the utilisation of yield strength of steel. The utilisation of steel will be maximum when the reduction factor will be 1 or close to one. According to winter formula, from slenderness of member, reduction factor can be calculated. With the help of this reduction factor effective cross-section (reduced cross-section) is calculated which is used for calculation of resistance of member. Therefore, according to this various plots are made to show the effect of thickness of core and thickness of faceplates on the reduction factor. Various graphs are plotted for different steel grades namely S355, S690 and S1100 as follows,

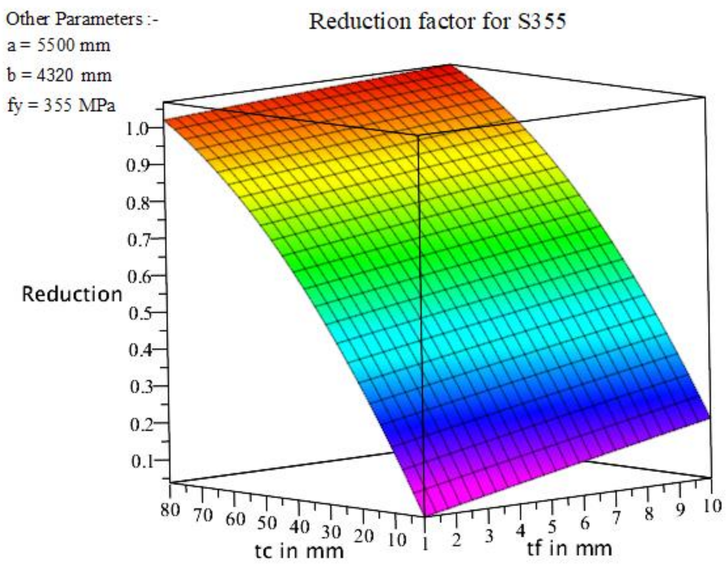


Figure 5.6 Reduction factor for a sandwich with faceplates S355

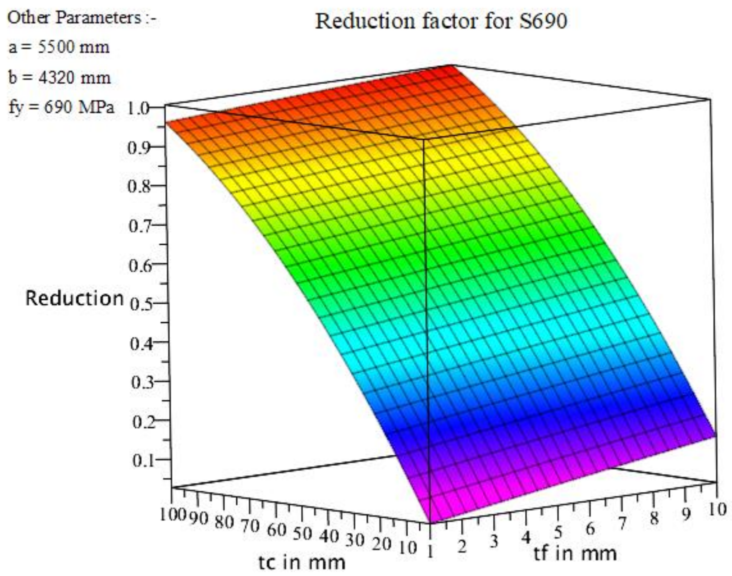


Figure 5.7 Reduction factor for a sandwich with faceplates S690

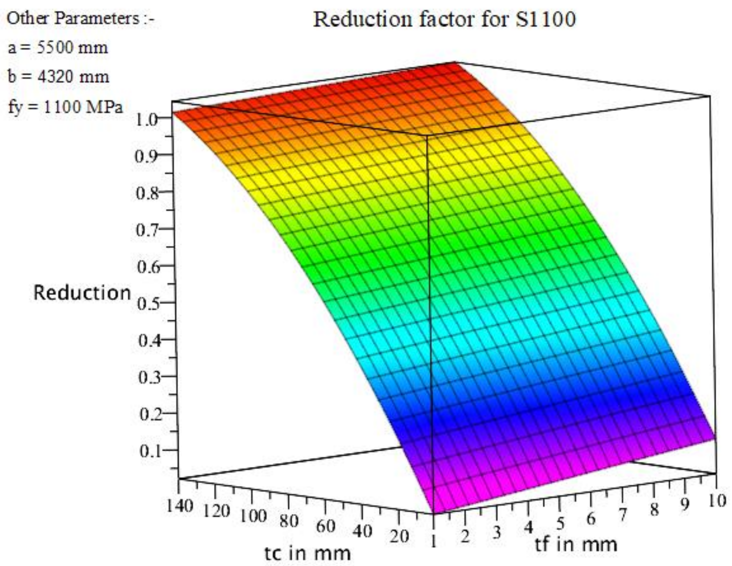


Figure 5.8 Reduction factor for a sandwich with faceplates S1100

All these graphs show common behaviour that, the thickness of core has a huge impact on the reduction factor whereas the thickness of faceplates has a really low effect on the reduction factor. This might be due to the fact that with an increase of core thickness, the distance between steel plate increases which results in an increase in distance between outer most fibres thereby increasing their utilisation and contribution. Also as per formula, flexural rigidity is proportional to the square of the thickness of core thickness. This also can be an explanation to, why core thickness has a large effect on buckling resistance.

After a certain limit, with an increase of core thickness reduction factor become one. This will indicate that now member will fail due to yield failure not due to buckling. If the reduction factor is 1 this means buckling is not issue any more but failure due to yielding is now governing.

According to this, from figures in section 4.4, we can guess that sandwich with core thickness 70 and 90 will be more optimised from material (steel) use viewpoint. However, at the same time, the weight of the sandwich panel is increased i.e. sandwich with core thickness 70 or 90 are not optimised in terms of weight reduction, weight reduction is decreased.

Therefore, in such cases, an optimised sandwich panel will be a bargain between both aims i.e. sandwich which fulfils both aims of weight reduction and material utilisation to a certain extent. The following table will show weight reduction and material utilisation achieved for the sandwich with other configurations.

Steel Grade	S355		S690		S1100	
Sandwich	4.3-70-4.3	4.1-90-4.1	2.8-70-2.8	2.3-90-2.3	2.1-70-2.1	1.7-90-1.7
Weight reduction	+16	+27	-7	-1	-18	-11
Steel utilisation	0.94	-	0.76	0.89	0.63	0.77

Table 5.1 Table for weight reduction & steel utilisation

In section 4.4.1, for sandwich configuration 4.3-70-4.3 weight reduction is not achieved but utilisation of steel is 0.94. For sandwich 2.7-90-2.7, weight reduction is not achieved but steel utilisation is more.

In section 4.4.2, sandwich configuration 2.8-70-28 gives a weight reduction of -7.3% and steel utilisation of 0.76. For sandwich 2.3-90-2.3, weight reduction is -1.2 % and steel utilisation of 0.89 can be observed.

In section 4.4.3, for sandwich panel with 2.1-70-2.1 configuration weight reduction of -18 % and steel utilisation of 0.63 is observed whereas sandwich configuration 1.7-90-1.7 results in a weight reduction of -11 % and steel utilisation of 0.77.

5.3 Results and Observations

- Optimization of sandwich panels were evaluated from two prospective, A. optimization to achieve maximum weight reduction and B. optimization to achieve maximum use of steel or faceplate material for buckling.
- Perspective of maximum weight reduction: - The thickness of the sandwich panel should be minimized especially the thickness of faceplates to achieve maximum weight reduction. Therefore, in this case, the use of extra high strength steel is justified. With extra high strength steel, both high buckling resistance and maximum weight reduction can be achieved.
- Perspective of maximum use of steel: - Utilization of faceplate material increases with an increase in distance between them. Therefore, to increase the utilization of steel in faceplates, the thickness of the core should be increased. But this results in an increase in weight of the sandwich panel. So higher the strength of steel, more thickness of core is needed to increase material utilization for buckling. Therefore, the use of extra high strength steel is not justified from a material usage point of view.
- Therefore, an optimized sandwich panel will be a bargain between weight reduction and material utilization.
- In considered example,
  - Sandwich with faceplates S355 gives steel utilization of 70 % but no weight reduction.
  - Sandwich with faceplates S690 gives steel utilization of 60 % & weight reduction of 9 %
  - Sandwich with faceplates S1100 gives steel utilization of 50 % & weight reduction of 20 %.

Therefore, it can be observed that with an increase in strength of steel, weight reduction increases but material utilization decreases.

6 Huisman Structure

6.1 Introduction to Drilling Tower

Drilling tower is an integrated system that is used for drilling wells such as oil or water wells on the subsurface of the earth. Drilling tower is massive structures used to support equipment used for oil or water well drilling, natural gas extraction wells. Drilling tower can be used for onshore or offshore application. They can be mounted on trucks, tracks or on land or marine-based structures such as floating oil platforms for offshore oil and gas extraction.

Drilling towers vary in size as per capacity. Small to medium-sized drilling tower usually have small drilling capacity and can be mobile enough. These are ones, which are used in mineral exploration drilling, water wells, blast hole, and environmental investigations. Increase in drilling capacity increases the size of the drilling tower. The larger tower is capable of drilling through thousands of meters of earth’s crust, using large mud pumps to circulate drilling mud, larger rigs with hug capacity, heavy drill bits, casing annulus for cooling and removing cutting, hoists in rig which can lift hundreds of tons of pipe. This all results in the requirement of the strong supporting structure, which can carry various loads such as regular loads, working load, environmental loads, etc. and still allow the structure to function properly and safely. Also, the mainframe of a structure should be cost-effective and optimized.

Huisman structure is drilling tower but with increased efficiency, safety along with multi-functioning capabilities. The structure will be explained in following coming section. Please refer to Huisman document “A18-42110-C1-001A Detailed Design Report MPT” and “A18-42110-C1-000B Basic Design Report MPT” for in-depth information & design.

6.2 Huisman Structure – Description

Huisman structure is a drilling tower, which can be used on offshore or onshore oil and gas extraction. Huisman designed this structure with the aim of drastically improving offshore drilling and equipment handling. This structure also explores new solutions to improve drilling operations.

It is a fully closed drilling tower, box girder type mast which houses all machinery inside. Box girder provides the main load-carrying function and an enclosed environment for all equipment. The tower is welded on a semi-submersible vessel. A main and aux well are serviced simultaneously; both sides use the same 600mt setback. Huisman structure is a stiffened plate octagonal tower manufactured with steel S355. Manipulators and trolley run on rails at the outside of the tower. These rails are made from steel S690 to deal with local wheel forces.

The tower includes two integrated crown blocks and rails for a travelling block on main and construction hoist side. The tower has two hoists; the main hoist is situated above well centre on the drilling floor. Construction hoist is situated above the centre of the construction floor on the other side of the tower. With two hoists many of drilling activities can be performed simultaneously, which increases efficiency.

Main hoist side is used to:

- Run telescopic joint
- Perform drilling activities
- Tripping drill string
- Land SSBOP or X-mas tree on a wellhead
- Perform completion activities

Construction hoist side is used to:

- Offline stand building
- Building surface casing strings
- Run marine drilling riser

Layout:-

Huisman structure (drilling tower) is a welded box girder type load-bearing structure with a relatively small footprint and a hoist on either side. All major equipment is mounted inside the tower in an enclosed environment, protecting it from the harsh, offshore environment. There is no lattice-type derrick structure around hoists creating open access to the well centre from three sides. This makes it possible to skid or hoist large objects directly to well centres.

Box structure provides an enclosed environment for the following equipment on the inside:

- E-cabinets
- Drawworks winches
- Passive heave compensation cylinders
- Pressure vessels

• Tuggers
• Auxiliary Equipment

The tower on the inside is considered as a non-hazardous area since there are no direct access doors to EX-zones and ventilation air is taken from a non-hazardous area.

Inside of the tower will be provided with cage ladders, stairs and platforms for safe and adequate access to equipment for operation, maintenance and service. There will also be doors and hatches to access platforms and walkways to equipment on the outside of the tower. The main access to the tower is provided through a vessel.

Standpipes for mud and cement are mounted on outside of tower as well as degasser vent line.

Hoist System:-
It's a device used for lowering or lifting a load with the help of a lift-wheel or drum around which chain or rope wraps. It can be operated manually or electrically or pneumatically driven and can use fibre, steel wire rope or chain as a lifting medium.

Both sides of the tower have their own independent hoist system. The main hoist is outfitted with a dual drum drawworks with passive heave compensation. Construction hoist consists of a single drum without heave compensation.
Hoist system comprises the following components:
• Travelling block, crown block and rigging
• Dual drum drawworks (main hoist only)
• Dual passive compensation cylinders (main hoist only)
• Power, control and safety system

Drawworks:-
Dual drum draw works is a system with two independent driven drums. This provides built-in redundancy due to the fact that if one winch fails, another winch can still operate travelling block at full load, however at half original speed.

Passive heave compensation:-
Passive heave compensation system delivers the necessary control required over long operating periods. The system balances the weight of the drill string, allowing for heave of the vessel. Two hydraulic cylinders integrated in the hoist system carry out passive heave compensation.

Tugger:-
Tugger is a device which is designed for marine towing operations for various types of ships including multi-purpose ship, oil field guard ship, an offshore support ship, etc. Typically, tugger winches are used to help move loads on deck. Pulling force normally ranges between 10-25 tonnes. Winches have a man riding capacity of 150 kg.

Pressure vessels:-

A pressure vessel is a container designed to hold gases or liquids at a pressure substantially different from the ambient pressure. Pressuring up is done with nitrogen supplied by vessels nitrogen generators. Pressuring down is done by venting nitrogen into open-air (environment). Pressure vessels are divided over multiple skids for working bottles and storage bottles. Pressure vessels are also used for the passive heave compensation system.

Huisman structure can be constructed on a floating platform or ship and used on the sea for oil gas extraction. The structure can be loaded in tension and compression. But an important type of load is compressive. Therefore, failure due to compression is one of the most important types of failure in this type of structure. There are five main sections of Huisman structure which are starting from the bottom, E-Room, Drawworks, Lower Heave, Upper Heave and Top Section.

Distribution of weight is shown below,

TABLE

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Table 6.1 Weight distribution of Heave section

6.3 Load and Load Combination

Occurring load on the structure can be categorized into regular, occasional and exceptional loads. Most of these loads speak for themselves, others will be shortly explained.

Regular loads

- Load due to own weight
- Operational load
- Out of plane influences

Occasional load

- Wind
- Vessel motions
- Expected Storm

Exceptional load

- Accidental heel/trim
- Accidental explosion
- Unexpected storm

General load combinations are,

- Regular load under normal operation without wind
- Regular load under normal operation combined with wind
- Regular loads combined with occasional and exceptional loads

Load cases (Load case matrix):-

Load Case	Load Case No.	Wind Speed (m/sec)	Manipulator Load (mt)	Setback Load (mt)	Hook Load (mt)	Hook Load (mt)	Deck Load (t/m^2)	Safety Factor
Static	1	-	10	600	820	0	5/3	1.67
	2	-	10	600	0	820	5/3	1.67
	3	-	10	600	820	180	5/3	1.67
	4	-	10	600	180	820	5/3	1.67
Operational Load	5	24.7	10	600	820	0	5/3	1.25
	6	24.7	10	600	0	820	5/3	1.25
	7	24.7	10	600	820	180	5/3	1.25
	8	24.7	10	600	180	820	5/3	1.25
Unexpected Storm	9	40.1	0	600	0	0	5/3	1.25
	10	40.1	0	600	0	0	5/3	1.25
Expected Storm	11	73.7	0	0	0	0	5/3	1.25
Accidental heel/trim	12	-	10	600	0	0	5/3	1
Accidental Explosion	13	24.7	10	600	820	0	5/3	1

Table 6.2 Load Case for heave section

Load Combinations:-

12 load cases explained above has various sub load case. Some of the load cases have subcases from A to H whereas some have subcases from A to D. These subcases is due to the fact that application of load can be from various directions. For example in case of wind, wind can be applied on structure from any direction mainly from 4 different direction which can be +x, -x, +z, -z. The same story is for expected and unexpected storm, transit and accidental load. Therefore, this gives rise to a total of 64 load combinations, which are applied to the structure.

A short explanation of load combination is given below,

1. First load combination is load cases with a functional load but no environmental loads. This includes the first four load cases which are static loads only but no environmental loads. Required safety factor for this part is 1.67.

2. Second load combination is load cases which consist of influence of regular load along with external environmental effects. This is considered in load case 5-8. Required safety factor for this part is 1.25. For environmental effects (wind and accelerations), eight sub-cases (a-h) are checked for each 45°.

3. Third part (LC9 - LC10) is unexpected storm condition. During this condition, the setback drum is full; there is insufficient time to empty it. It is assumed that there is sufficient time to put trolley loads in the rotary table. Therefore, trolley loads are zero. All travelling equipment is down.

4. Fourth part (LC11) is the expected storm. This is a hurricane in the Gulf of Mexico. The vessel is anchored and waits for the storm to pass. All functional loads are zero. There is sufficient time to empty setback drum.

5. LC12 is the accidental heel of 27°. Either heel or trim is checked. Environmental effects are all zero. It is assumed that there is sufficient time to lower trolley loads into rotary tables. There is insufficient time to empty setback. The required safety factor is 1.00; this means that structure should survive. Plastic analysis is allowed.

6. LC13 is an explosion load case. The pressure is assumed to be 200 MPa. E-room section wall should withstand pressure due to explosion.

From the basic design report the critical load case for the heave section is operational LC5c. Therefore, LC5a through LC5h are checked. The maximum setback eye forces occur in LC9, therefore this LC is checked also. Accidental heel/trim (LC12) and transport (LC17) are checked because these give maximum Fy and Fx respectively in the PHM eyes. Please refer to Huisman document “A18-42110-C1-001A Detailed Design Report MPT” for in-depth information.

6.4 Heave Section

Heave section is selected for study since for lower sections of Huisman structure fatigue is determining and therefore not fit for a scope of this case study. The middle section of the structure is heave section. Lower and upper heave section is an octagonal shell type structure. Shell is constructed out of different section each with the stiffened plate. The section consists out of 8 steel plates which are cut into required dimensions and then welded together. Weight of heave section is approximately 15.94% of the weight of the total structure. This chapter will evaluate the potential for weight reduction using the sandwich panel. It is stated that buckling is the main concern when designing heave section. Some redesigned are analysed considering these aspects and compared with the original.

Original design of section:-

Global layout and dimensions of the heave section are shown below,

FIGURE

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Figure 6.1 Lower and Upper Heave Section

In the original design, the heave section is made up of S355 stiffened steel plate. Transverse ring stiffeners are placed at 5500 mm distance centre to centre. At certain positions, this distance is varied which is a result of a mechanical requirement. For example at that position certain equipment is placed, which results in the demand for ring stiffener. Total weight of heave section is 76 tonnes. This contributes to approximately 15.94 % of the total weight of the structure. Decreasing weight of heave section, by using the sandwich panels, will result in a lower reaction force on lower and supporting structure, which is quite important in offshore applications.

6.5 Calculation

IMEP spreadsheet will be used for checking static strength of section. IMEP is an excel spreadsheet which is internally developed by Huisman Equipment to determine main section forces. FEM will be used to evaluate static and buckling checks. Dimensions of the sandwich panel are selected in such a way that it will satisfy both weight reduction and strength criteria resulting in optimised design. For choosing a sandwich with proper dimensions previous chapter 4 will be referred, in which comparison of the stiffened plate is done with the sandwich panel. The compared stiffened plate is one which is used in heave section. Therefore, chapter 4 can give an appropriate approximation of the dimensions of sandwich panels.

From the IMEP spreadsheet, most decisive load cases for heave section are found out. These load cases are,

Load Case	Load Combination	Description
5	7,5-C	Operational Load and environmental effect
11	46, 11-B	Expected Storm

Table 6.3 Decisive load cases

Main section forces for a selected sandwich panel at two sections i.e. lower and upper heave section are calculated with the help of IMEP sheet and is arranged in the following table,

Cross-section	Load Combination	Main Section Forces					
		Fu(KN)	Fv(KN)	Fw(KN)	Mu(KN-m)	Mv(KN-m)	Mw(KN-m)
A-A	46	-8047	-6248	0	-192	628	-128248
B-B	7	-16630	0	2657	11549	-112318	2343

Table 6.4 Main section forces for decisive load cases and cross-section

Above table shows that load case 46 is critical of section A-A for all considered sandwich panels. Load combination 7 is critical for section B-B for all considered sandwich panels.

6.5.1 Sandwich with Faceplates S355

From chapter 4.4.1, it can be observed that the selected sandwich shows the same buckling strength as that of the stiffened plate. So same sandwich configuration will be used in designing of heave section.

Static strength:-

Main section forces are used to determine static buckling strength for each section.

Cross-section	Load Combination	Max. stress	Safety factor	Required safety factor
A-A	7	150	2.4	1.25
A-A	46	155	2.3	1.25
B-B	7	105	3.3	1.25
B-B	46	70	4.9	1.25

Table 6.5 Main section forces for a sandwich with steel S355

Weight:-

Comparison of weight is done in the following table,

		Original Design	New Design
Global Dimension	Height	28400 mm	28400 mm
	Width (Perimeter)	29240.4 mm	29240.4 mm
	Stiffened	Yes	No
Steel Grade		S355	S355
Steel Weight		82000 Kg	70000 Kg
Weight of Aluminium Foam		-	29000 Kg
Total Weight		82000 Kg	99000 Kg (+21%)
Number of Parts		46	8

Table 6.6 Weight comparison for a sandwich with steel S355

FEM Result:-

The chosen sandwich configuration is also validated using finite element method. Both buckling stress and Von Mises equivalent stress are calculated. Eigenvalue buckling analysis is done on the structure to find load multiplication factor. Load multiplication factor will give the value of load or stress at which structure will buckle. As per the theory, the first buckling mode is critical in the case of flexural buckling analysis. Generally in Eigen buckling analysis, it is customary to calculate at least 10 buckling mode but load multiplier value corresponding to the first mode is only important from point of view of flexural buckling. Also, it is important to find out the value of stress at the buckling load. For this minimum principal stress is calculated.

Buckling Stress:-

In ANSYS analysis, load multiplication factor can give a general idea about buckling of structure. Load multiplication factor cannot directly give a buckling load or stress at buckling. But it can be used to calculate buckling stress and also as an indicator & can be used for the purpose of comparison. The structure having a lower value of load multiplier will buckle at a smaller load as compared to the structure with higher load multiplier. The following figure shows the first Eigen buckling mode and load multiplier corresponding to it.

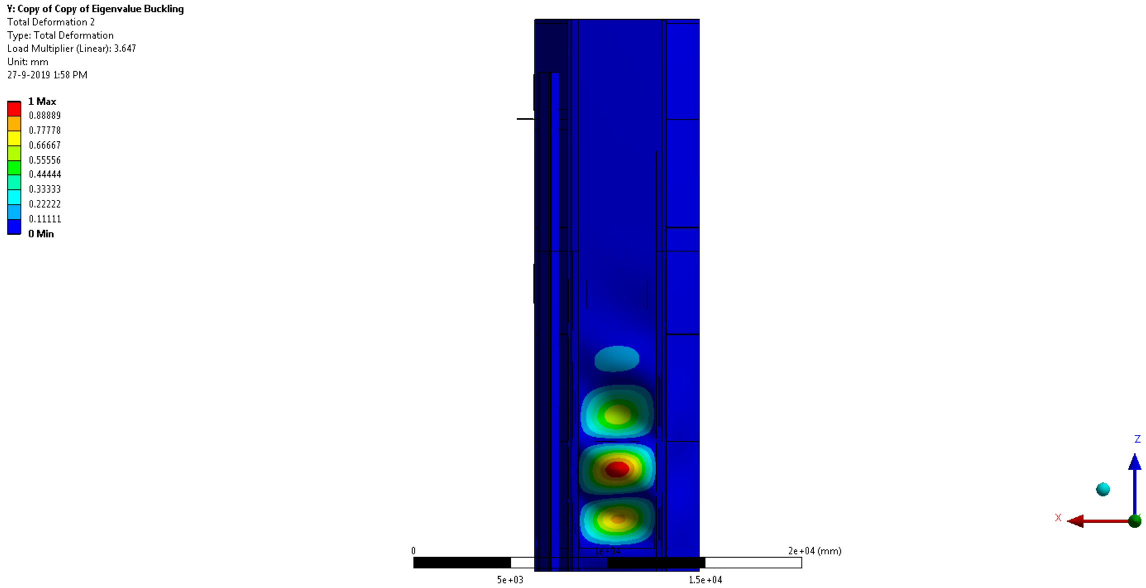

Figure 6.2 Buckling of heave section

From the above figure, it can be observed that load multiplier for the above structure is,

$$Load\ Multiplier\ =\ 3.6$$

Now, from the above figure position of maximum displacement can be observed. After this, minimum principal stress at the same position will be measured. In practice it is difficult to point out this position exactly, therefore some points near the point of maximum deformation are arbitrarily chosen and, minimum principal stress at all these points will be measured. The following figure will give an idea of this.

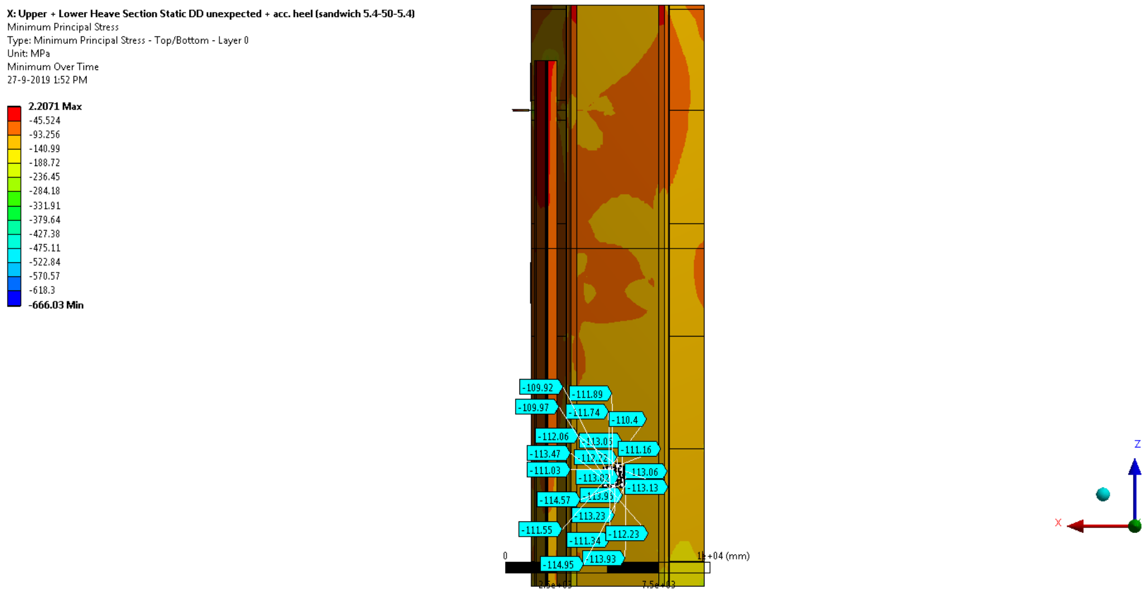

Figure 6.3 Minimum principle stress

The above figure shows minimum principal stress at various points close to the point of maximum deflection observed in Eigen buckling analysis. Next step would be to calculate the average of all these values. Average of these values is,

$$Minimum\ principle\ stress\ =\ -112.4\ MPa$$

Form these two value i.e. load multiplier and minimum principal stress, stress at buckling can be calculated as follows,

$$Stress\ at\ buckling = load\ multiplier * minimum\ principle\ stress$$
$$Stress\ at\ buckling = 3.6 * -112.4$$
$$Stress\ at\ buckling = -404.5\ MPa$$

<u>Von Mises stress:-</u>

Von Mises equivalent stress can be evaluated by doing a static check of structure. Von Mises stress can be compared with a yield stress of material to determine the static strength of the structure. In ANSYS Von Mises equivalent stress can be calculated for all load cases and represented in one figure. The following figure shows the distribution of Von Mises stress for Huisman structure.

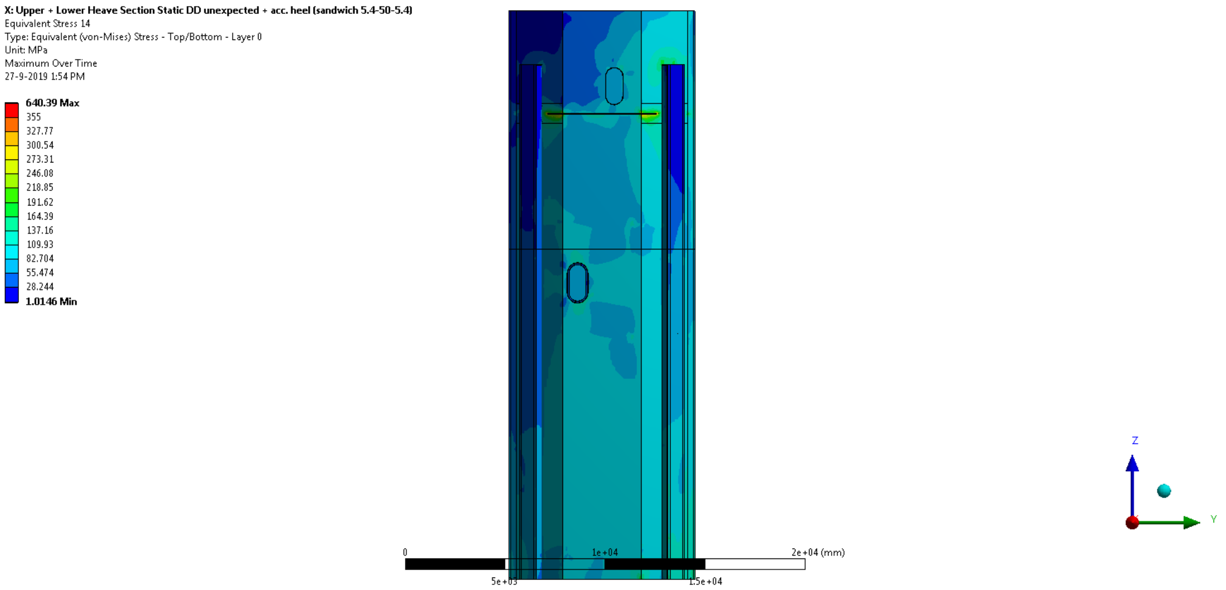

Figure 6.4 Von Mises stress (front view)

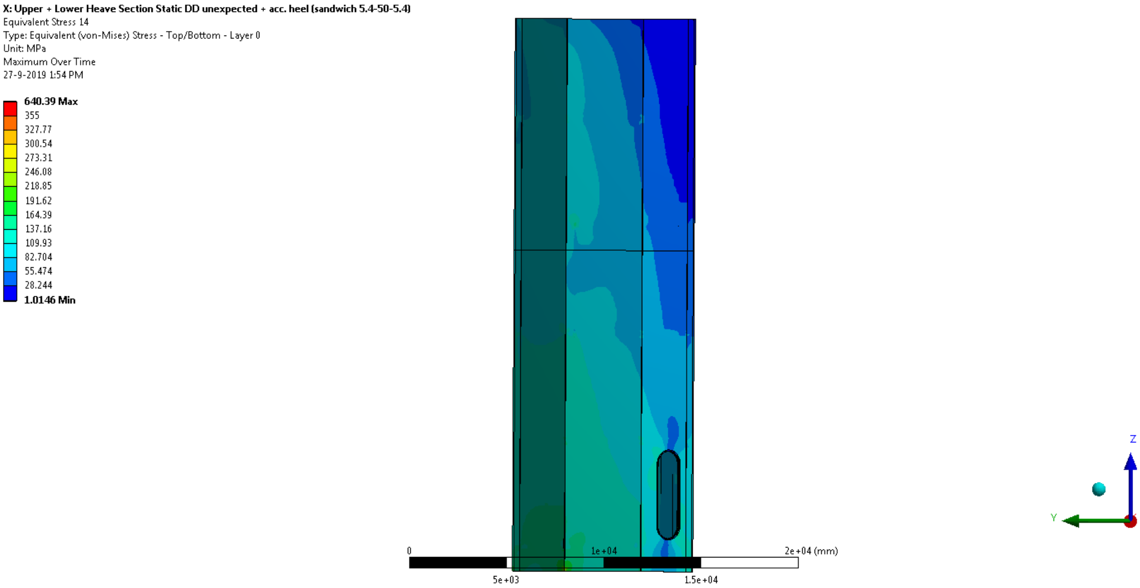

Figure 6.5 Von Mises stress (back view)

Figure 6.4 & Figure 6.5 gives an overall stress distribution. It can be observed that stress distribution over the majority of places is below the yield of material. But in some places, it is more than that of the yield of material used. Please refer to the following the figure,

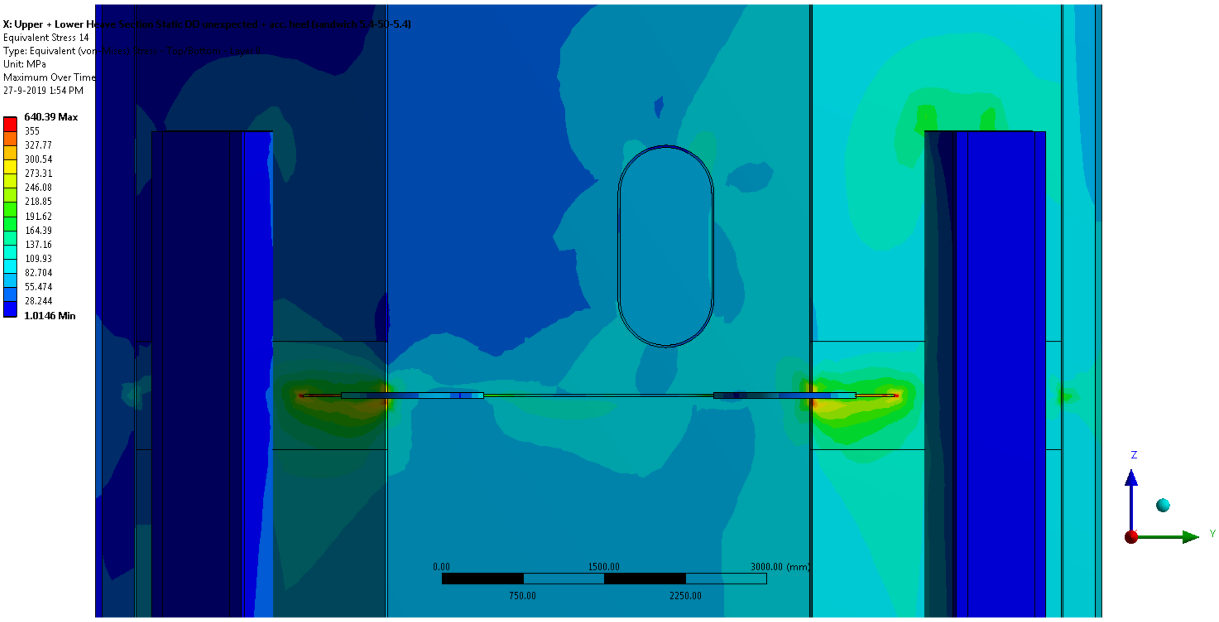

Figure 6.6 Von Mises stresses at joints

Figure 6.6 shows that Von Mises stress at joints is more than that of the yield of material. Therefore, at such position local yielding of material is possible. But this local yielding is confined to a small area. Also, if martial yield at these points then local stresses will be redistributed. So due to these two facts i.e. confined to small area and redistribution of stress, this stress concentration can be neglected. So in end, it can be concluded that utilisation of material is high since stress at buckling is higher than the yield strength of the material. Also, structure satisfies static check. Therefore, this concludes that design is strong enough for occurring forces.

Minimum principle stress:-

Minimum Principle stress can be evaluated with the help of FEM. Minimum principle stress calculated with FEM can be compared with a yield stress of material to determine the static strength of the structure. In ANSYS, minimum principal stress can be calculated for all load cases and represented in one figure. The following figure shows the distribution of the minimum principle for Huisman structure.

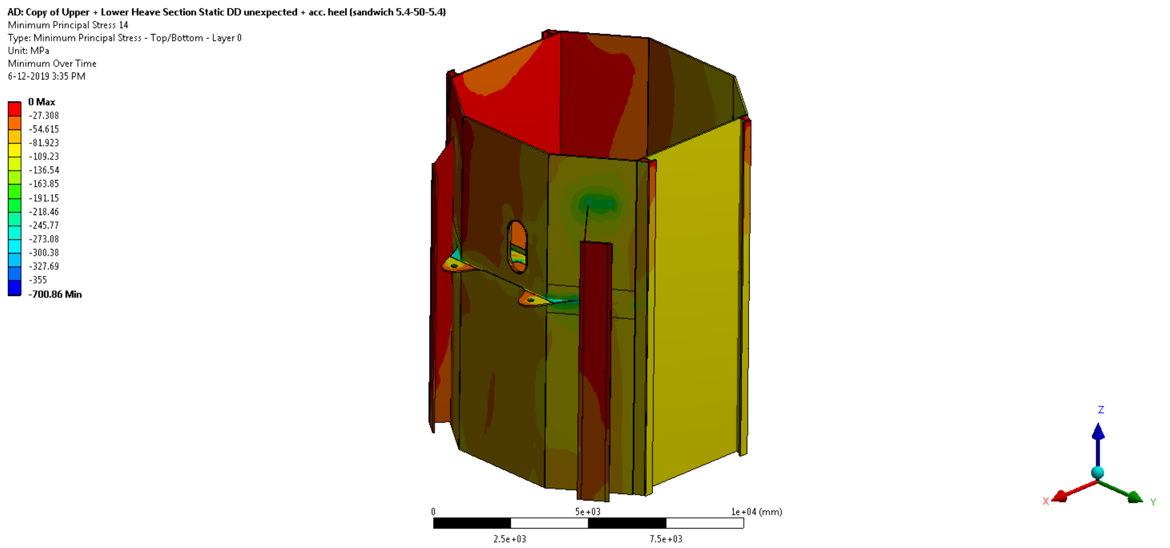

Figure 6.7 Minimum principle stress (upper part)

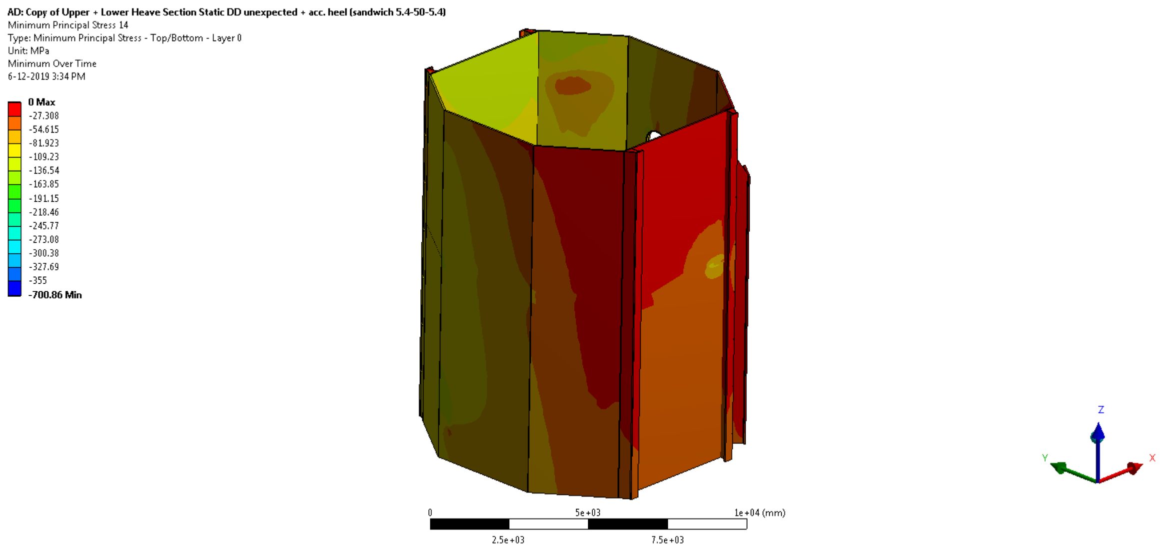

Figure 6.8 Minimum principle stress (upper part)

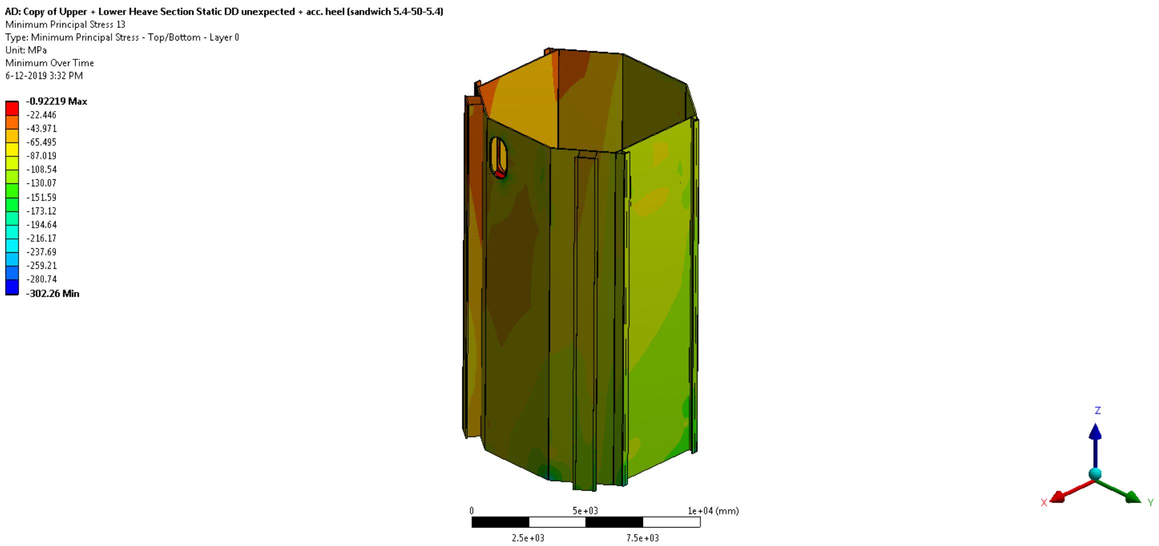

Figure 6.9 Minimum principle stress (lower part)

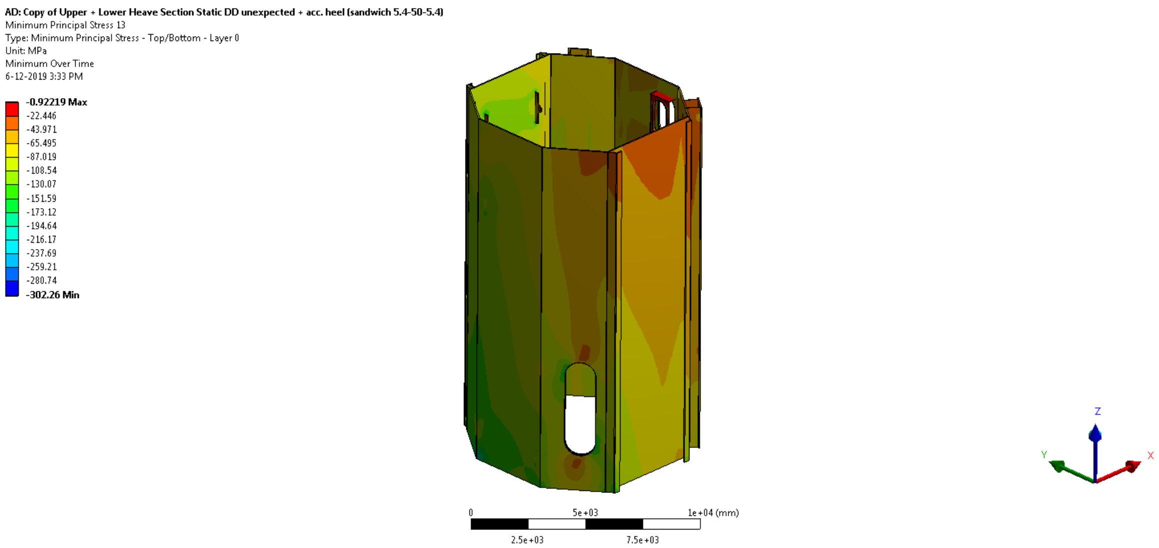

Figure 6.10 Minimum principle stress (lower part)

Above figures show minimum principal stresses at various points close to cross-section AA and BB. This Minimum principle stress calculated with FE Analysis can be compared with results calculated with analytical analysis in Table 6.5. From this comparison, it can be observed that values of stresses calculated with both methods are close to each other.

6.5.2 Sandwich with Faceplates S690

From chapter 4.4.2, it can be observed that the selected sandwich shows the same buckling strength as that of the stiffened plate. So same sandwich configuration will be used in designing of heave section.

Static strength:-

Main section forces are used to determine static buckling strength for each section.

Cross-section	Load Combination	Max. stress	Safety factor	Required safety factor
A-A	7	190	3.7	1.25
A-A	46	215	3.2	1.25
B-B	7	130	5.3	1.25
B-B	46	100	6.9	1.25

Table 6.7 Main section forces for a sandwich with steel S690

Weight:-

Comparison of weight is done in the following table,

		Original Design	New Design
Global Dimension	Height	28400 mm	28400 mm
	Width (Perimeter)	29240.4 mm	29240.4 mm
	Stiffened	Yes	No
Steel Grade		S355	S690
Steel Weight		82000 Kg	47000 Kg
Weight of Aluminium Foam		-	29000 Kg
Total Weight		82000 Kg	76000 Kg (-8%)
Number of Parts		46	8

Table 6.8 Weight comparison for a sandwich with steel S690

FEM Result:-

The chosen sandwich configuration is also validated using finite element method. Both buckling stress and Von Mises equivalent stress are calculated. Eigenvalue buckling analysis is done on the structure to find load multiplication factor. Load multiplication factor will give the value of load or stress at which structure will buckle. As per the theory, the first buckling mode is critical in the case of flexural buckling analysis. Generally, in Eigen buckling analysis, it is customary to calculate at least 10 buckling mode but load multiplier value corresponding to the first mode is only important from point of view of flexural buckling. Also, it is important to find out the value of stress at the buckling load. For this minimum principal stress is calculated.

Buckling Stress:-

In ANSYS analysis, load multiplication factor can give a general idea about buckling of structure. Load multiplication factor cannot be directly give buckling load or stress at buckling. But it can be used to calculate buckling stress and also as an indicator & can be used for the purpose of comparison. The structure having a lower value of load multiplier will buckle at smaller load as compared to the structure with higher load multiplier. The following figure shows the first Eigen buckling mode and load multiplier corresponding to it.

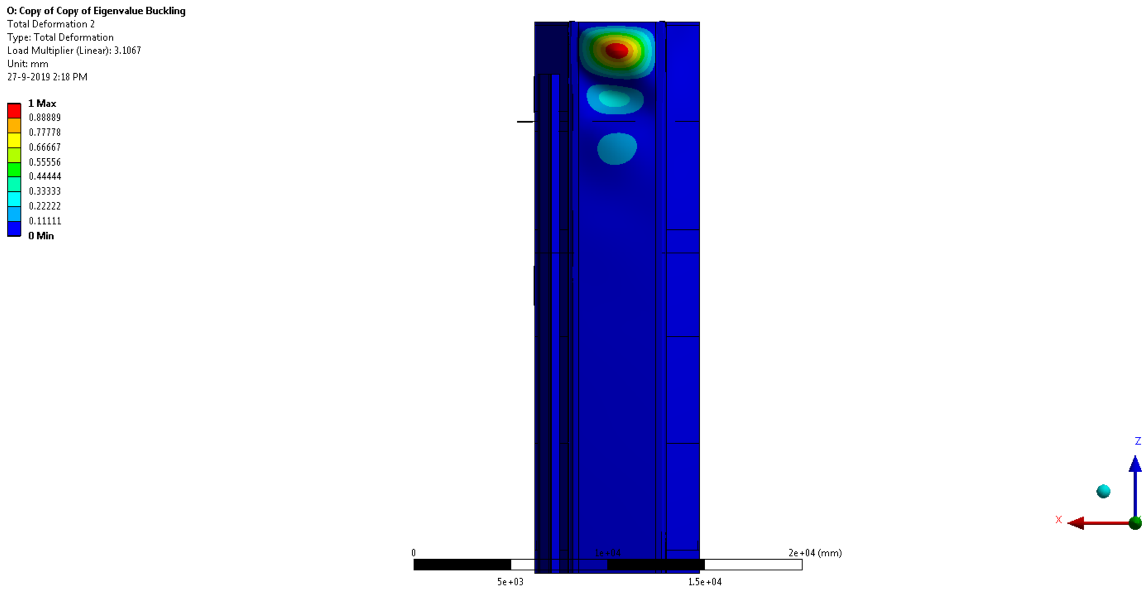

Figure 6.11 Buckling of heave section

From the above figure, it can be observed that load multiplier for the above structure is,

$$Load\ Multiplier\ =\ 3.11$$

Now, from the above figure position of maximum displacement can be observed. After this, minimum principal stress at the same position will be measured. In practice it is difficult to point out this position exactly, therefore some points near the point of maximum deformation are arbitrarily chosen and, minimum principal stress at all these points will be measured. The following figure will give an idea of this.

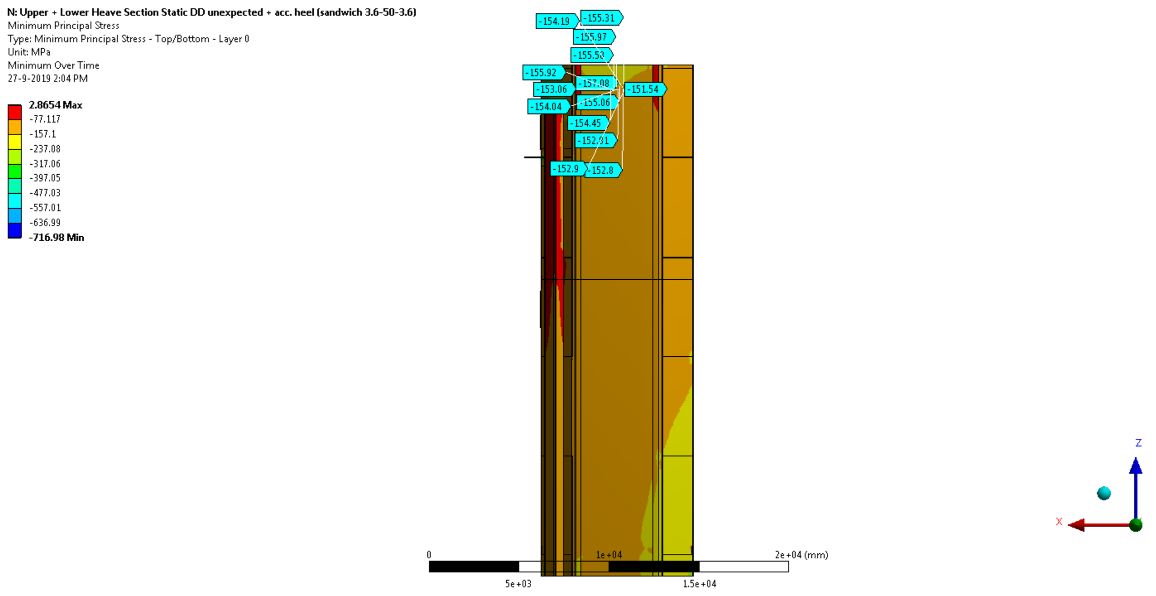

Figure 6.12 Minimum principle stress

The above figure shows minimum principal stress at various points close to the point of maximum deflection observed in Eigen buckling analysis. Next step would be to calculate the average of all these values. Average of these values is,

$$Minimum\ principle\ stress\ =\ -154.4\ MPa$$

Form these two value i.e. load multiplier and minimum principal stress, stress at buckling can be calculated as follows,

$$Stress\ at\ buckling = load\ multiplier * minimum\ principle\ stress$$
$$Stress\ at\ buckling = 3.11 * -154.4$$
$$Stress\ at\ buckling = -479.99\ MPa$$

Von Mises stress:-

Von Mises equivalent stress can be evaluated for doing a static check of structure. Von Mises stress can be compared with a yield stress of material to determine the static strength of the structure. In ANSYS Von Mises equivalent stress can be calculated for all load cases and represented in one figure. The following figure shows the distribution of Von Mises stress for Huisman structure.

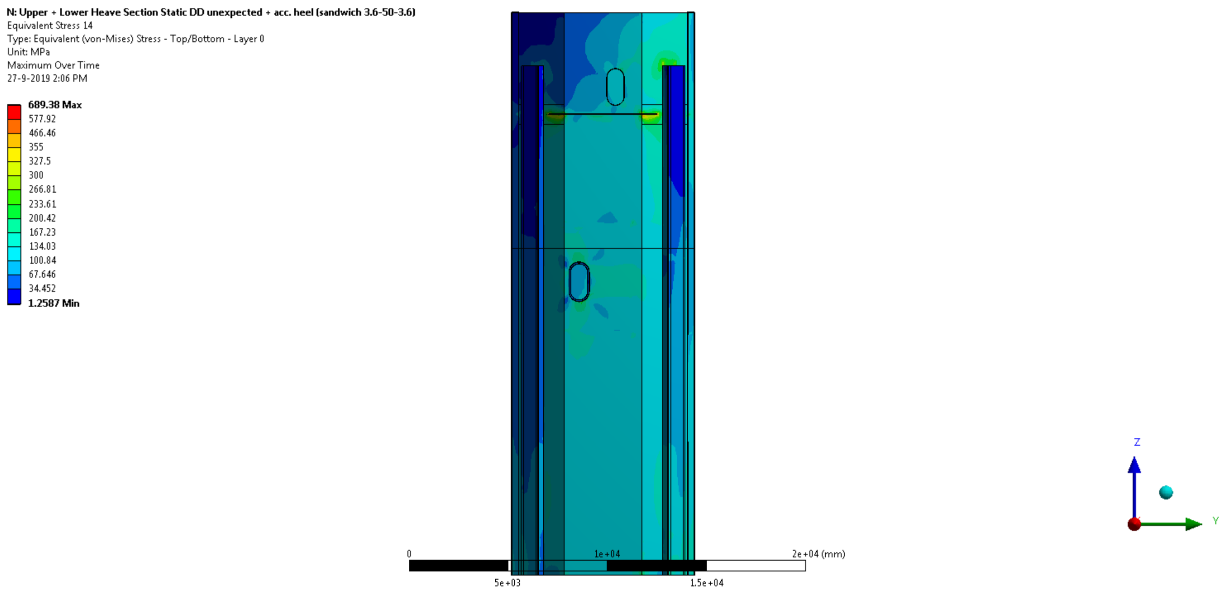

Figure 6.13 Von Mises stress (front view)

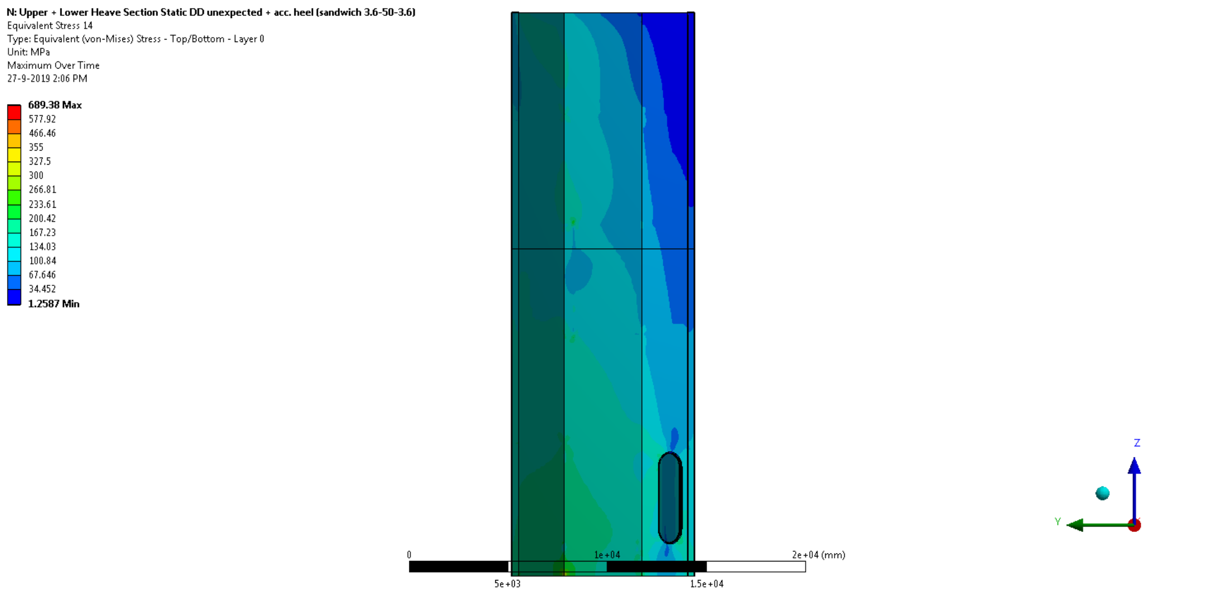

Figure 6.14 Von Mises stress (back view)

Figure 6.9 & Figure 6.10 gives an overall stress distribution. It can be observed that stress distribution over the majority of places is below the yield of material. But in some places, it is more than that of the yield of material used. Please refer to the following the figure,

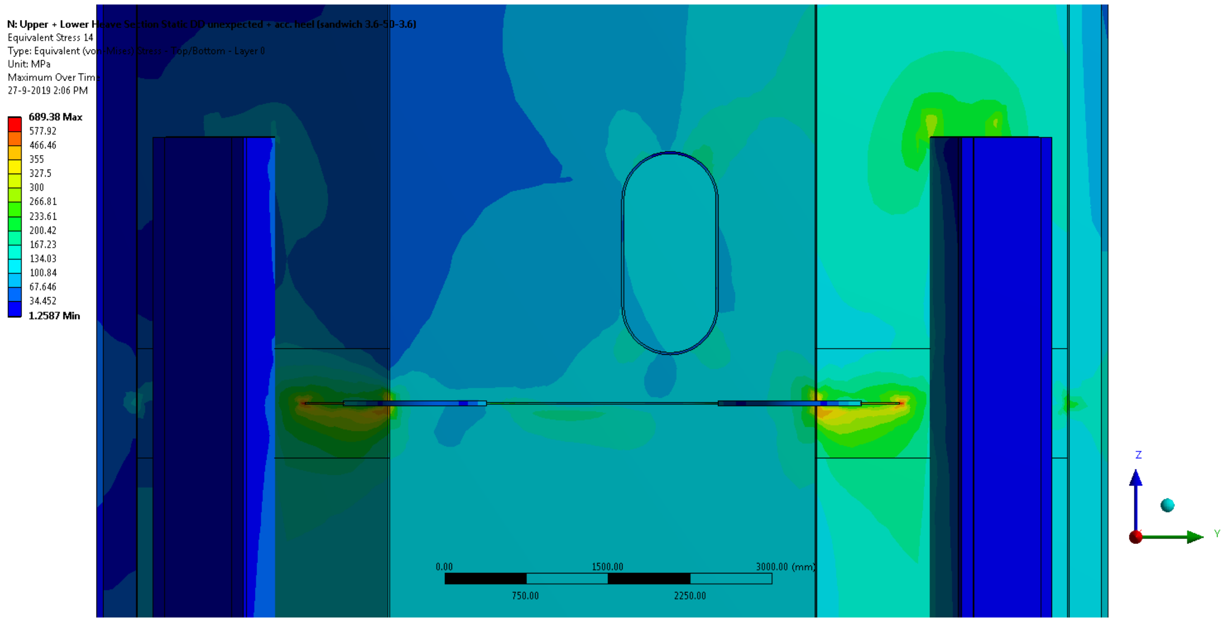

Figure 6.15 Von Mises stresses at joints

Figure 6.11 shows that Von Mises stress at joints is more than that of the yield of material. Therefore, at such position local yielding of material is possible. But this local yielding is confined to a small area. Also, if martial yield at these points then local stresses will be redistributed. So due to these two facts i.e. confined to small area and redistribution of stress, this stress concentration can be neglected.

So in end, it can be concluded that utilisation of material is high since stress at buckling is higher than the yield strength of the material. Also, structure satisfies static check. Therefore, this concludes that design is strong enough for occurring forces.

Minimum principle stress:-

Minimum Principle stress can be evaluated with the help of FEM. Minimum principle stress calculated with FEM can be compared with a yield stress of material to determine the static strength of the structure. In ANSYS, minimum principal stress can be calculated for all load cases and represented in one figure. The following figure shows the distribution of the minimum principle for Huisman structure.

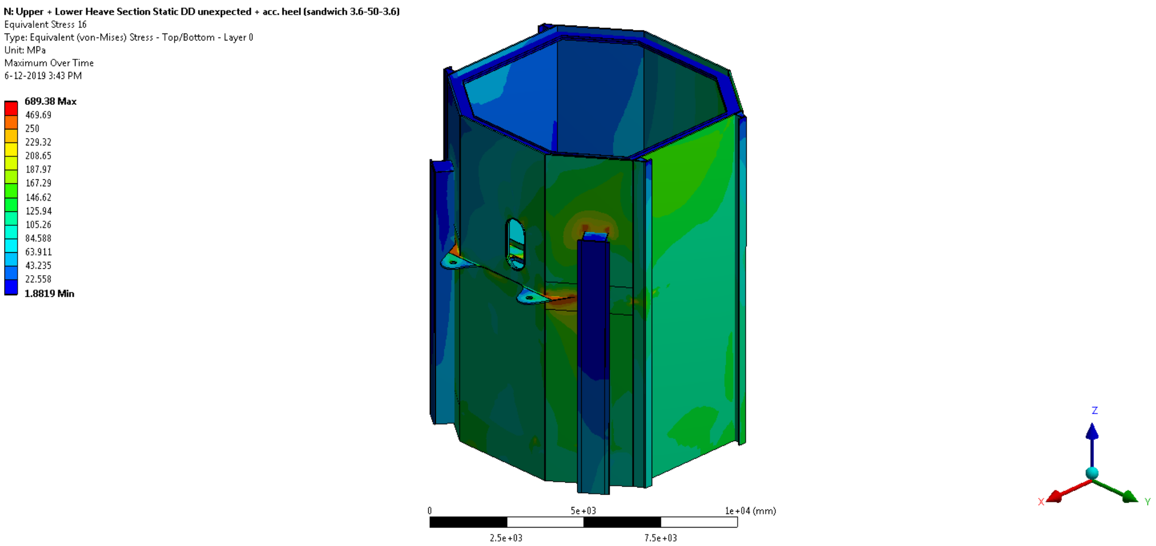

Figure 6.16 Minimum principle stress (upper part)

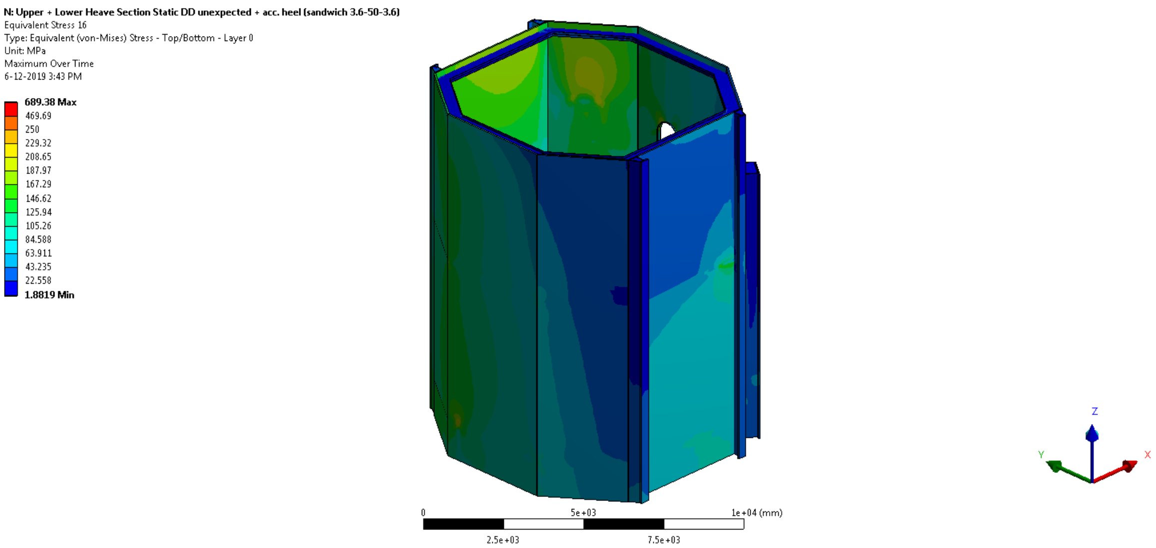

Figure 6.17 Minimum principle stress (upper part)

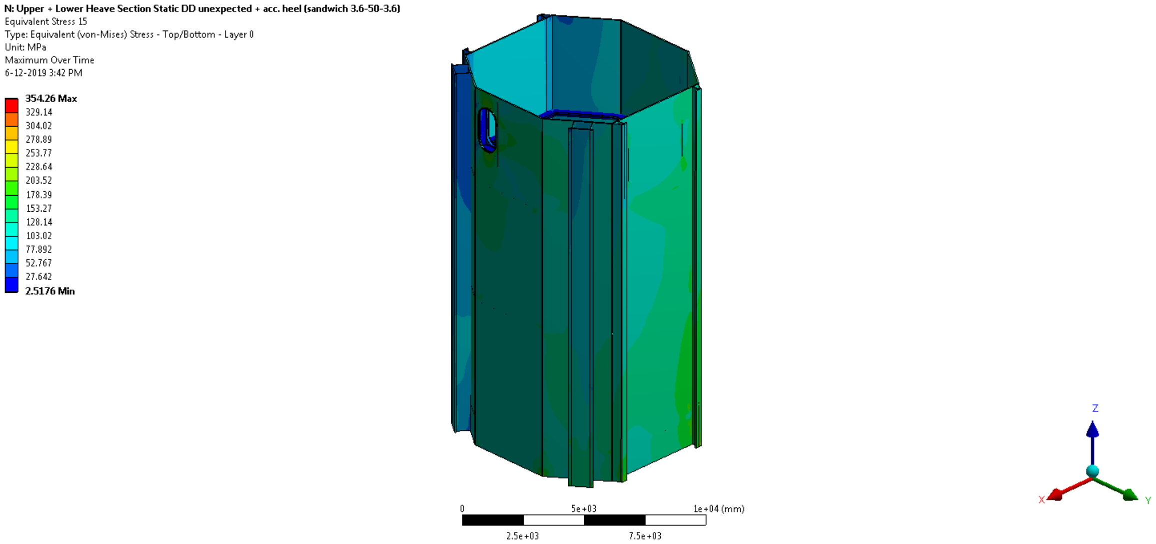

Figure 6.18 Minimum principle stress (lower part)

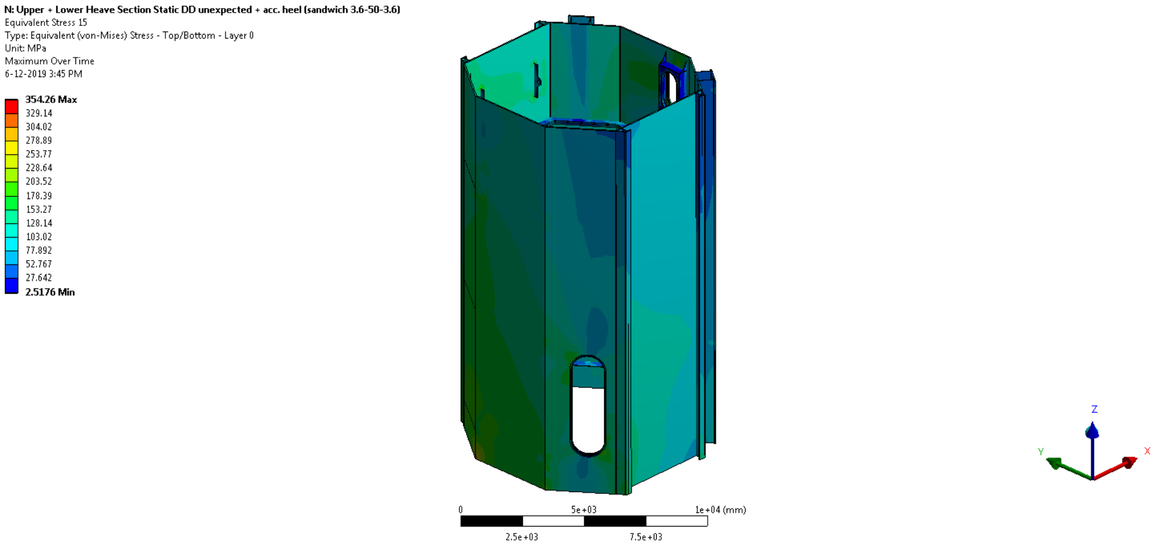

Figure 6.19 Minimum principle stress (lower part)

Above figures show minimum principal stresses at various points close to cross-section AA and BB. This Minimum principle stress calculated with FE Analysis can be compared with results calculated with analytical analysis in Table 6.7. From this comparison, it can be observed that values of stresses calculated with both methods are close to each other.

6.5.3 Sandwich with Faceplates S1100

From chapter 4.4.3, it can be observed that the selected sandwich shows the same buckling strength as that of the stiffened plate. So same sandwich configuration will be used in designing of heave section.

Static strength:-

Main section forces are used to determine static buckling strength for each section.

Cross-section	Load Combination	Max. stress	Safety factor	Required safety factor
A-A	7	220	5.1	1.25
A-A	46	260	4.3	1.25
B-B	7	150	7.4	1.25
B-B	46	125	9.0	1.25

Table 6.9 Main section forces for a sandwich with steel S1100

Weight:-

Comparison of weight is done in the following table,

		Original Design	New Design
Global Dimension	Height	28400 mm	28400 mm
	Width (Perimeter)	29240.4 mm	29240.4 mm
	Stiffened	Yes	No
Steel Grade		S355	S1100
Steel Weight		82000 Kg	37000 Kg
Weight of Aluminium Foam		-	29000 Kg
Total Weight		82000 Kg	66000 Kg (-20 %)
Number of Parts		46	8

Table 6.10 Weight comparison for a sandwich with steel S1100

FEM:-

The chosen sandwich configuration is also validated using finite element method. Both buckling stress and Von Mises equivalent stress are calculated. Eigenvalue buckling analysis is done on the structure to find load multiplication factor. Load multiplication factor will give the value of load or stress at which structure will buckle. As per the theory, the first buckling mode is critical in the case of flexural buckling analysis. Generally in Eigen buckling analysis, it is customary to calculate at least 10 buckling mode but load multiplier value corresponding to the first mode is only important from point of view of flexural buckling. Also, it is important to find out the value of stress at the buckling load. For this minimum principal stress is calculated.

Buckling Stress:-

In ANSYS analysis, load multiplication factor can give a general idea about buckling of structure. Load multiplication factor cannot be directly give buckling load or stress at buckling. But it can be used to calculate buckling stress and also as an indicator & can be used for the purpose of comparison. The structure having a lower value of load multiplier will buckle at smaller load as compared to the structure with higher load multiplier. The following figure shows the first Eigen buckling mode and load multiplier corresponding to it.

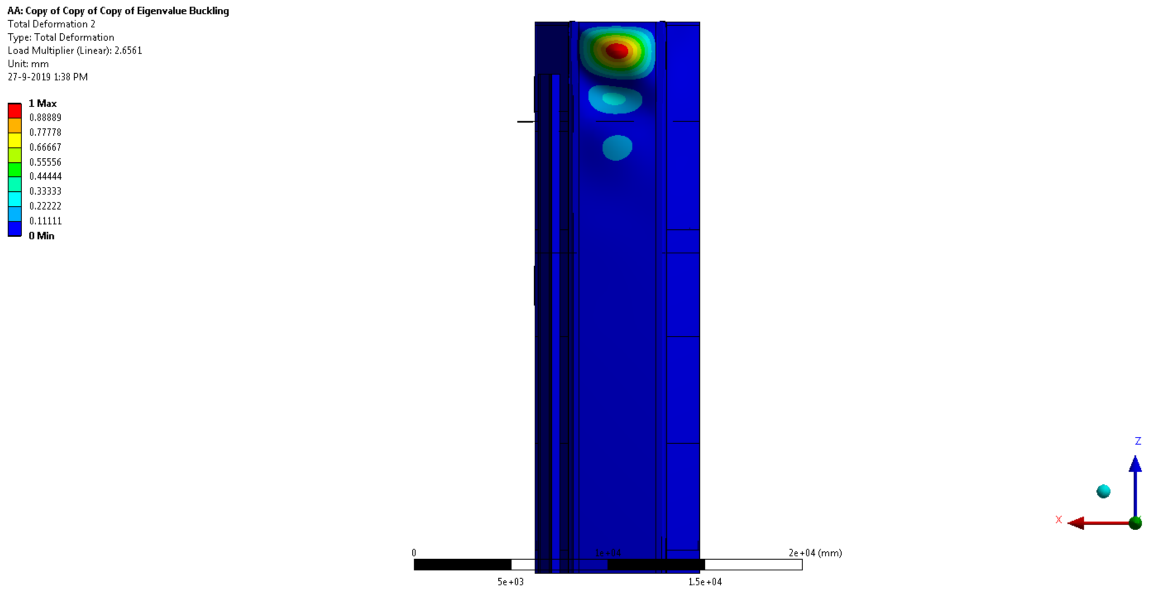

Figure 6.20 Buckling of heave section

From the above figure, it can be observed that load multiplier for the above structure is,

$$Load\ Multiplier\ =\ 2.65$$

Now, from the above figure position of maximum displacement can be observed. After this, minimum principal stress at the same position will be measured. In practice, it is difficult to point out this position exactly, therefore some points near the point of maximum deformation are arbitrarily chosen and, minimum principal stress at all these points will be measured. The following figure will give an idea of this.

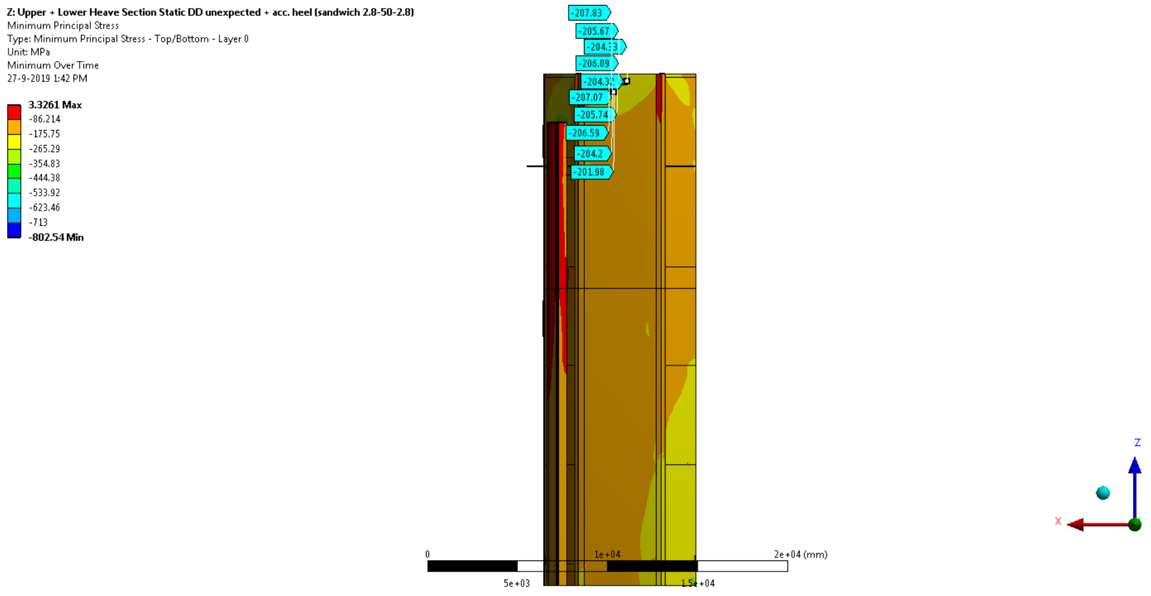

Figure 6.21 Minimum principle stress

The above figure shows minimum principal stress at various points close to the point of maximum deflection observed in Eigen buckling analysis. Next step would be to calculate the average of all these values. Average of these values is,

$$Minimum\ principle\ stress\ =\ -205.4\ MPa$$

Form these two value i.e. load multiplier and minimum principal stress, stress at buckling can be calculated as follows,

$$Stress\ at\ buckling = load\ multiplier * minimum\ principle\ stress$$
$$Stress\ at\ buckling = 2.65 * -205.4$$
$$Stress\ at\ buckling = -544.3\ MPa$$

Von Mises stress:-

Von Mises equivalent stress can be evaluated for doing a static check of structure. Von Mises stress can be compared with a yield stress of material to determine the static strength of the structure. In ANSYS Von Mises equivalent stress can be calculated for all load cases and represented in one figure. The following figure shows the distribution of Von Mises stress for Huisman structure.

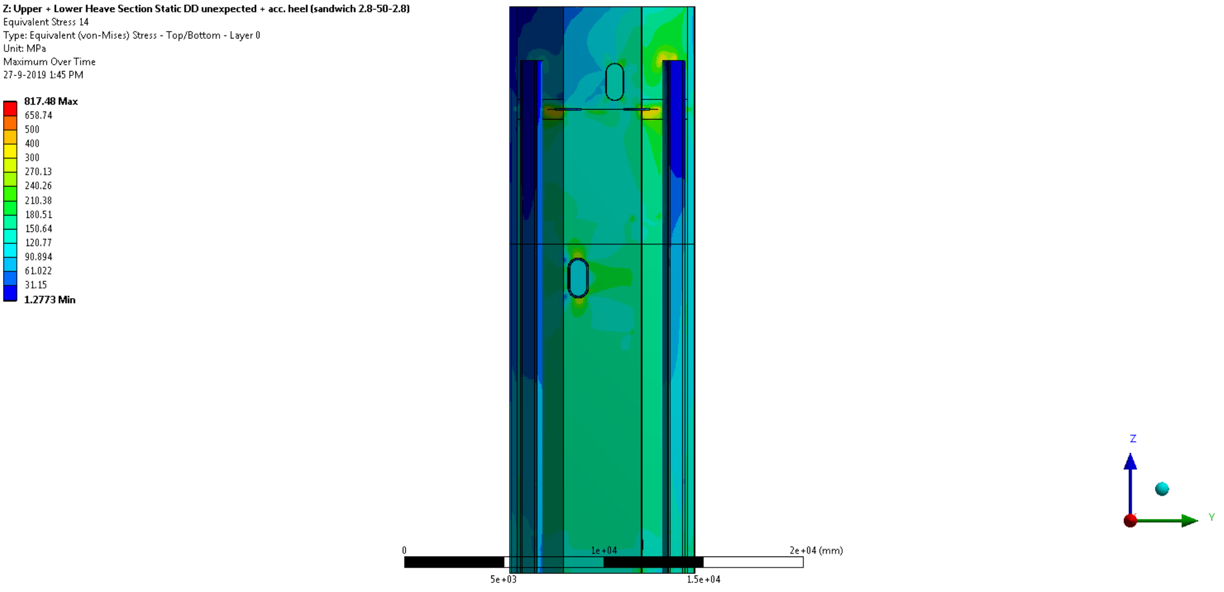

Figure 6.22 Von Mises stress (front view)

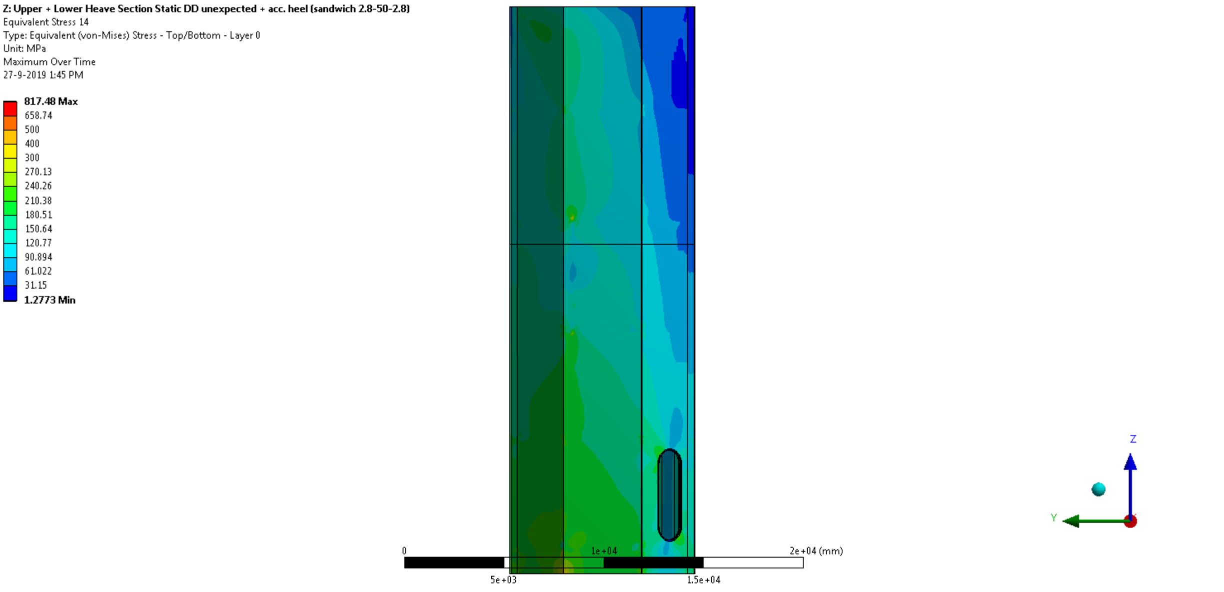

Figure 6.23 Von Mises stress (back view)

Figure 6.14 & Figure 6.15 gives an overall stress distribution. It can be observed that stress distribution over the majority of places is below the yield of material. But in some places, it is more than that of the yield of material used. Please refer to the following figure,

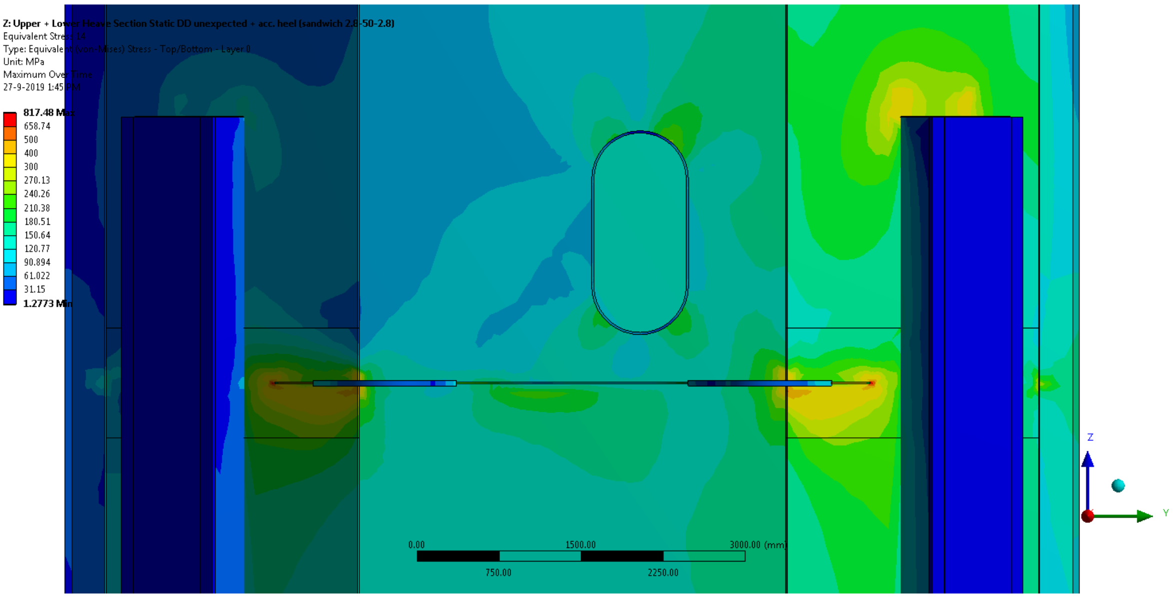

Figure 6.24 Von Mises stresses at joints

Figure 6.16 shows that Von Mises stress at joints is more than that of the yield of material. Therefore, at such position local yielding of material is possible. But this local yielding is confined to a small area. Also, if martial yield at these points then local stresses will be redistributed. So due to these two facts i.e. confined to small area and redistribution of stress, this stress concentration can be neglected.

So in end, it can be concluded that utilisation of material is high since stress at buckling is higher than the yield strength of the material. Also, structure satisfies static check. Therefore, this concludes that design is strong enough for occurring forces.

Minimum principle stress:-

Minimum Principle stress can be evaluated with the help of FEM. Minimum principle stress calculated with FEM can be compared with a yield stress of material to determine the static strength of the structure. In ANSYS, minimum principal stress can be calculated for all load cases and represented in one figure. The following figure shows the distribution of the minimum principle for Huisman structure.

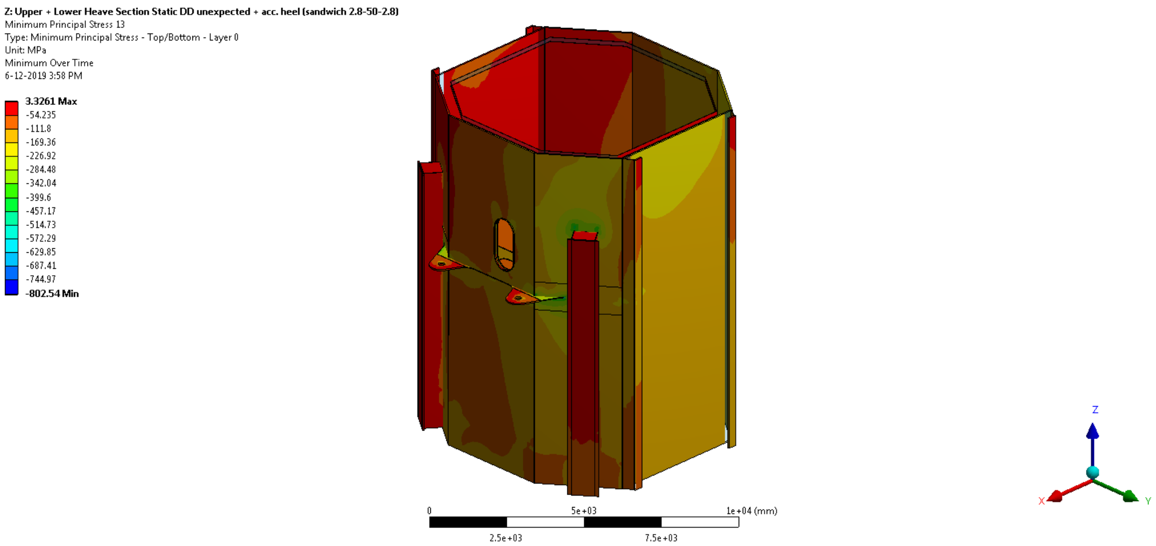

Figure 6 25 Minimum principle stress (upper part)

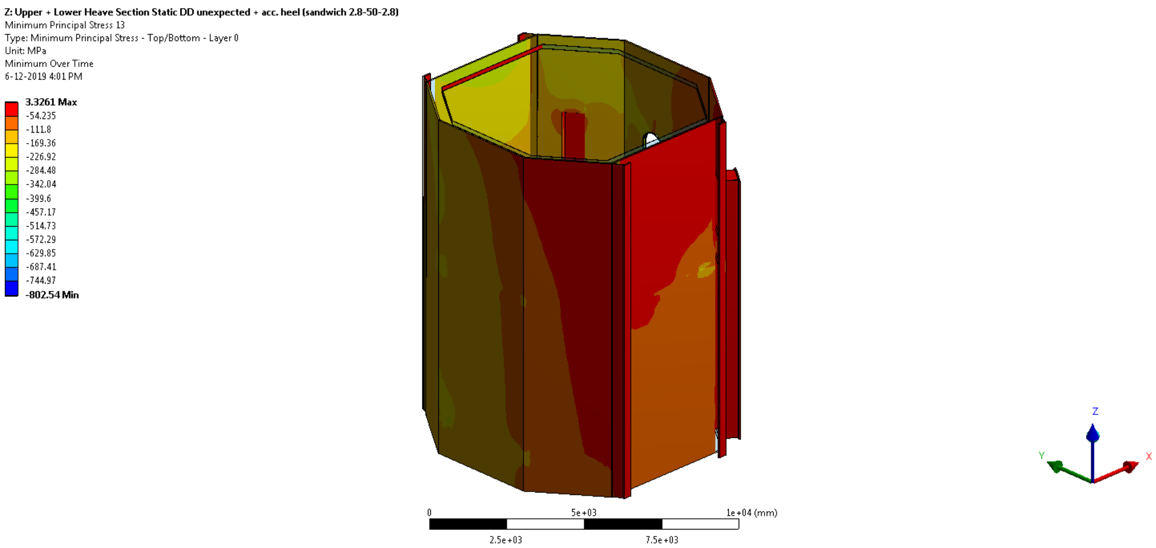

Figure 6.26 Minimum principle stress (upper part)

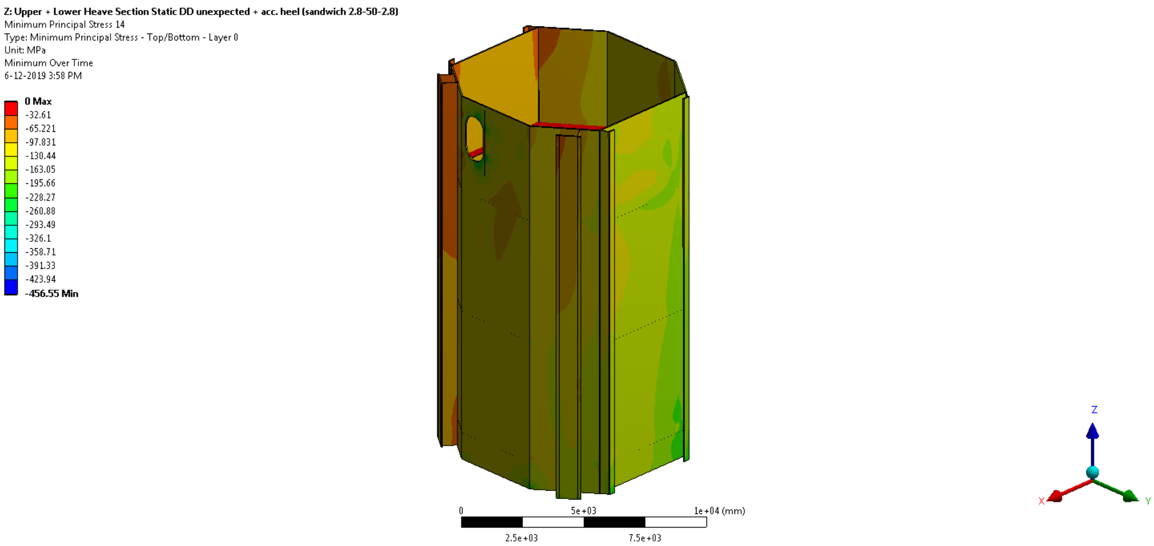

Figure 6.27 Minimum principle stress (lower part)

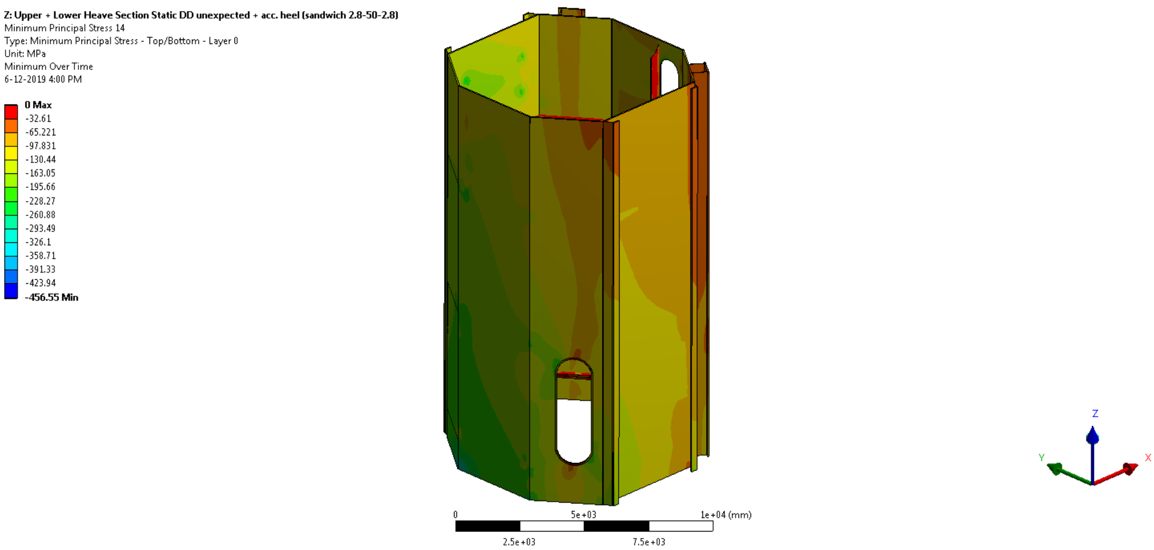

Figure 6.28 Minimum principle stress (lower part)

Above figures show minimum principal stresses at various points close to cross-section AA and BB. This Minimum principle stress calculated with FE Analysis can be compared with results calculated with analytical analysis in Table 6.9. From this comparison, it can be observed that values of stresses calculated with both methods are close to each other.

6.6 Cost Comparison

In the above chapter, heave section of Huisman structure is designed with the help of the sandwich panels. The main aim of using sandwich panels is the self-weight reduction. Therefore, from above section 0, we can observe that the weight reduction of heave section is achieved with the help of sandwich panels.

Along with weight reduction, it is important that new material should also be cost-effective. It is important that the traditional solution and new solution should be comparable from an economic point of view. Therefore, in this section, the cost of a heave section built with stiffened panels and sandwich panels are compared. In this section, first, the cost of a heave section built with stiffened plate will be calculated, afterwards, the cost of a heave section built with the sandwich panels will be calculated. Finally, both cost will be compared to determine which solution is economical. The main aim of this section is to investigate that if sandwich panels are used for weight reduction, it also gives an economical solution. Cost assessment will be performed by including material cost and cost for welding.

6.6.1 Cost of Original Design

Cost of material:-

In the original design of the heaves section, stiffened plates are used. The thickness of the base plate and thickness & width of stiffeners used in heave section is uniform over the entire length. Only the number of stiffeners used for stiffening varies as per requirement. Heave section is an octagonal shell-type structure consisting of stiffened plates of various dimensions as shown in Figure 6.1. This octagonal shape can be represented in another rectangular form as in the following Figure 6.17.

FIGURE

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Figure 6.29 Heave section with Stiffened Plate

In the above figure, solid lines represent dimensions of the base plate and vertical dotted lines represent stiffeners. From the above figure, we can determine the amount of material used in the manufacturing of heave section. Amount of steel used in heave section is a summation of steel used in base plate and steel used in stiffeners.

For cost estimation following values are considered,

- The cost of steel is 0.85 Euro per Kg.
- Also, the cost of cutting should be considered in the estimate. Cost of cutting is approximated as, 25% of 85 cents which is 21.25 cents for every 1000 Kg of steel.
- The density of steel used in 7850 Kg/m^3.

All these considered values (cost of steel and cost of cutting) are according to Huisman Equipment.

The following table will give the cost of the base plate,

Length (mm)	28400
Width (mm)	29240
Thickness (mm)	10
Volume (mm^3)	8.3×10^9
Weight (Kg)	65000
Cost of steel (Euro)	55000
Cost of cutting (Euro)	14
Total Cost (Euro)	55400

Table 6.11 Cost of the base plate

Similarly, the following table will give the cost of stiffeners,

Length (mm)	28400
Width (mm)	200
Thickness (mm)	10
Number of stiffeners	38
Volume (mm^3)	2.2×10^9
Weight (Kg)	17000
Cost of steel (Euro)	14000
Cost of cutting (Euro)	4
Total Cost (Euro)	14400

Table 6.12 Cost of stiffeners

Cost of welding:-

Stiffeners used for the stiffening of the base plate are welded to the base plate. All connection between stiffened plates are welded connections. The time required for welding dictates prices of welding. The time required for welding depends on the amount of welding. Therefore, the amount of welding has a considerable impact on the final cost. For the connection between the base plate and stiffener, weld on both sides is used. Only one weld is considered for connection between stiffened plates (base plates).

For cost estimation following values are considered,

- The pace of welding is 3 hr/m i.e. for finishing weld of 1 meter 3 hours are required.
- Cost of welding is 55 Euros per hour

All these considered values are according to Huisman Equipment. So the cost of welding can be calculated as follows,

Cost of Weld							
Length	Number of welds	Total length of weld (mm)	Total length of weld (metres)	Time for welding (hr/m)	Total time required (hr)	Cost for welding (Euro per hr)	Total cost (Euro)
5500	336	1848000	1800	3	5500	55	304900
5200	84	436800	440	3	1300	55	72100
1200	84	100800	100	3	300	55	16600
5000	10	50000	50	3	150	55	8300
2860	20	57200	60	3	170	55	9400
3900	10	39000	40	3	120	55	6400
Total			2500		7600		417700

Table 6.13 Cost of welding of heave section with the stiffened plate

The total cost of heave section is a summation of cost of the base plate, cost of stiffener and cost of welding.

Cost of base plate (Euro)	*Cost of stiffeners (Euro)*	*Cost of weld (Euro)*	*Total Cost (Euro)*
55400	14400	417700	488000

Table 6.14 Cost of the original design of heave section

Therefore, the preliminary cost of heave section manufactured with stiffened plate is € 488 000 /-

6.6.2 Cost of New Design

In the modified design of heave section sandwich panels are used. The following figure shows the replacement of stiffened plate with sandwich panels in heave section.

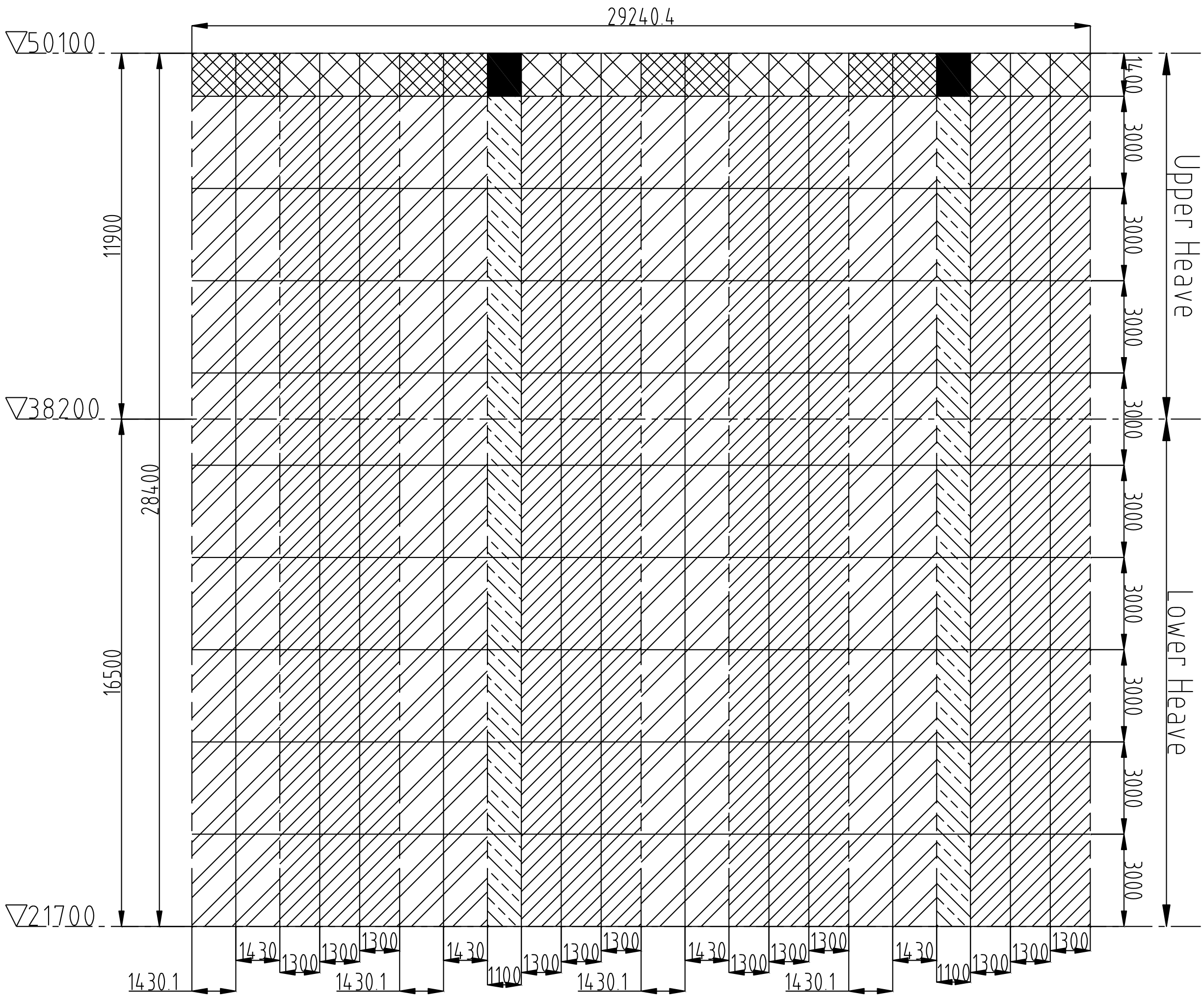


Figure 6.30 Heave section with sandwich panel

In the above figure, different hatch patterns are used to highlight the sandwich panel of different dimensions. Identical patterns represent sandwich with identical dimension. Solid lines represent boundaries of sandwich panels as well as weld length between sandwich panels.

Sandwich panels are prefabricated panels of specific dimensions and configuration. Specific companies manufacture these panels. Cost of sandwich panels used in the design is requested from Havel Metal Foam. Fabrication cost of sandwich panel decreases with an increase in the amount of sandwich panel. Also, fabrication cost decreases if sandwich panels are identical i.e. large number of the sandwich panels having the same dimensions are manufactured. So from this point of view above arrangement is chosen such that it will increase the number of sandwich panels with the same dimensions. One more reason behind using the above type of arrangement is fabrication limits. Havel Metal Foam cannot produce sandwich with dimensions (size) more than 3000 mm by 1500 mm. It can be observed from the above figure that all sandwich panels used in new design have dimensions less than the production limits stated by Havel Metal Foam.

Heave section is designed with sandwich S355, S690 and S1100. Arrangement of sandwich panels is exactly the same for all three sandwich panels. Only their configuration (thickness) varies as per grade of steel. Therefore, for all three sandwich panels amount of weld and the cost of the weld will be the same only the manufacturing cost of the sandwich panel will vary.

Cost of welding:-

All connection between sandwich panels is welded connections. Amount of welding has a considerable impact on the final cost. One side weld is considered for connection between sandwich panels.

For cost estimation following values are considered,

- The pace of welding is 3 hr/m i.e. for finishing weld of 1 meter 3 hours are required.
- Cost of welding is 55 Euros per hour

All these considered values are according to Huisman Equipment. Therefore, the cost of welding can be as follows,

Cost of Weld							
Length	Number of welds	Total length of weld (mm)	Total length of weld (metres)	Time for welding (hr/m)	Total time required (hr)	Cost for welding (Euro per hr)	Total cost (Euro)
3000	198	594000	590	3	1800	55	98000
1400	22	30800	30	3	90	55	5100
1430	72	102960	100	3	310	55	17000
1300	108	140400	140	3	420	55	23200
1100	18	19800	20	3	60	55	3300
Total			890		2700		146500

Table 6.15 Cost of welding of sandwich panel

Cost of sandwich with faceplates S355:-

For S355 sandwich of configuration, 5.4-50-5.4 is used. Its cost is requested from Havel Metal Foam. According to which cost of the used sandwich panel is,

Sandwich (5.4-50-5.4)				
Length (mm)	Width (mm)	Number of Panels	Cost per panel (euro)	Total Cost (euro)
3000	1430	72	2453	176600
3000	1300	108	2230	240800
3000	1100	18	2154	38800
1400	1430	8	1469	11700
1400	1300	12	1188	14300
1400	1100	2	1130	2300
Total				484400

Table 6.16 Cost of Sandwich S355

The total cost of heave section is a summation of the cost of sandwich panels S355 and cost of welding.

Cost of Sandwich Panel S355 (Euro)	*Cost of weld (Euro)*	*Total cost (Euro)*
484400	146500	631000

Table 6.17 Cost of Heave section with Sandwich S355

Therefore, the cost of a heave section designed with sandwich S355 is € 631 000 /-

Cost of sandwich with faceplates S690:-

For S690 sandwich of configuration, 3.6-50-3.6 is used. Its cost is requested from Havel Metal Foam. According to which cost of the used sandwich panel is,

Sandwich (3.6-50-3.6)				
Length (mm)	Width (mm)	Number of Panels	Cost per panel (euro)	Total Cost (euro)
3000	1430	72	2374	170900
3000	1300	108	2158	233100
3000	1100	18	2093	37700
1400	1430	8	1432	11500
1400	1300	12	1154	13900
1400	1100	2	1101	2200
Total				469200

Table 6.18 Cost of Sandwich Panels

Havel Metal Foam does not manufacture sandwich with extra high strength steel. Therefore, cost received from Havel is the cost of the sandwich with the same configuration as requested but manufactured with steel S355. This cost can be used for cost estimation but small modification will be required. In the modification, cost foam will be calculated by deducting the cost of faceplates of S355 from fabrication cost received from Havel. Afterwards, the cost of faceplates of S690 will be added to the cost of foam so as to get the cost of the desired sandwich panel. This will give a rough estimation of the design cost of sandwich panel S690. The estimated cost is shown below,

Cost of Sandwich S355 (Euro)	Cost of Steel S355 (Euro)	Cost of Foam (Euro)	Cost of S690 (Euro)	Cost of Sandwich S690 (Euro)
469200	39900	429300	56300	485600

Table 6.19 Cost of Sandwich S690

The total cost of heave section is a summation of the cost of sandwich panels S690 and cost of welding.

Cost of Sandwich Panel S690 (Euro)	*Cost of weld (Euro)*	*Total cost of Heave Section (Euro)*
485600	146500	632000

Table 6.20 Cost of Heave section with Sandwich S690

Therefore, the cost of a heave section designed with sandwich S690 is € 632 000 /-

Cost of sandwich with faceplates S1100:-

For S1100 sandwich of configuration, 2.8-50-2.8 is used. Its cost is requested from Havel Metal Foam. According to which cost of the used sandwich panel is,

Sandwich (2.8-50-2.8)				
Length (mm)	Width (mm)	Number of Panels	Cost per panel (euro)	Total Cost (euro)
3000	1430	72	2323	167300
3000	1300	108	2112	228100
3000	1100	18	2069	37200
1400	1430	8	1426	11400
1400	1300	12	1141	13700
1400	1100	2	1097	2200
Total				459900

Table 6.21 Cost of sandwich panels used in the design

Havel Metal Foam does not manufacture sandwich with extra high strength steel. Therefore, cost received from Havel is the cost of the sandwich with the same configuration as requested but manufactured with steel S355. This cost can be used for cost estimation but small modification will be required. In the modification, cost foam will be calculated by deducting the cost of faceplates of S355 from fabrication cost received from Havel. Afterwards, the cost of faceplates of S1100 will be added to the cost of foam so as to get the cost of the desired sandwich panel. This will give a rough estimation of the design cost of the sandwich panel. The estimated cost is shown below,

Cost of Sandwich S355 (Euro)	Cost of Steel S355 (Euro)	Cost of Foam (Euro)	Cost of S1100 (Euro)	Cost of Sandwich S1100 (Euro)
459900	31000	428800	73000	501800

Table 6.22 Cost of Sandwich S1100

The total cost of heave section is a summation of the cost of sandwich panels S1100 and cost of welding.

Cost of Sandwich Panel S1100 (Euro)	*Cost of weld (Euro)*	*Total cost (Euro)*
501800	146500	648000

Table 6.23 Cost of Heave section with Sandwich S1100

Therefore, the cost of a heave section designed with sandwich S1100 is € 648 000 /-

Finally, all these calculated costs can be arranged properly in a table and can be compared to determine whether the sandwich panel is a comparable economical solution to stiffened plates.

	Cost Comparison			
Type of Sandwich Panel	Heave with Sandwich Panel (€)	Heave with Stiffened Plate (€)	Difference (€)	Change in cost compared to Stiffened Plate (%)
Sandwich with S355	631000	488000	143000	+ 29
Sandwich with S690	632000	488000	144000	+ 30
Sandwich with S1100	648000	488000	160000	+ 33

Table 6.24 Cost Comparison between Stiffened Plate and Sandwich Panels

Form the above table we can observe that the cost of heave section designed with the sandwich panel is higher than the cost of the original design. The main reason behind this cost increment is due to the fact that the manufacturing cost of a sandwich panel is higher than the cost of a stiffened plate. On the contrary, one important observation is that the amount and cost of welding have reduced significantly.

6.7 Results and Observations

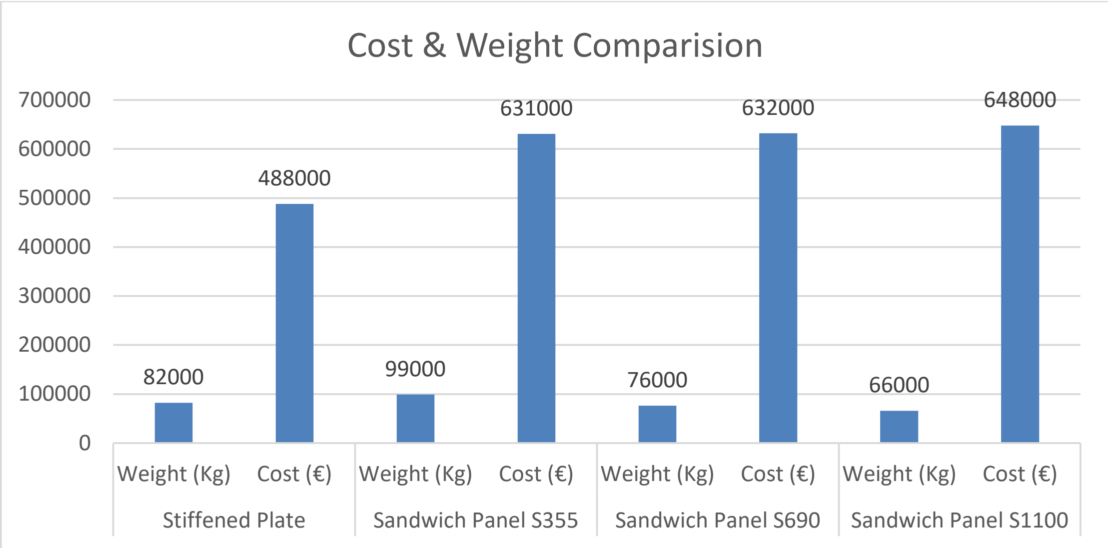


Figure 6.31 Cost & weight Comparison

Weight:-

- Use of high strength steel S355 for faceplate does not result in weight reduction. To achieve weight reduction use of extra high strength steel for faceplates is required.
- Sandwich with faceplates S355 fulfils buckling and static strength requirements but the requirement of weight reduction is not fulfilled. In this case, weight of the resulting structure, new design, is increased by 21 % as compared to the original design.
- Sandwich with faceplates S690 fulfils buckling and static strength requirements along with the requirement of weight reduction. Weight of the resulting structure, new design, is reduced by 8% as compared to the original design.
- Sandwich with faceplates S1100 results in an increase in weight reduction even further. With S1100 sandwich the highest weight reduction is achieved along with the required buckling and static strength. Weight of the resulting structure, new design, is reduced by 20 % as compared to the original design.

Cost Comparison:-

- Cost of heave section manufacture with stiffened plate comes out to be 488000 Euros. Cost of heave section designed with a sandwich as compared to a stiffened plate is,
  - increased by around 29 % than the original cost, for a sandwich with S355 faceplates
  - increased by around 30 % than the original cost, for a sandwich with S690 faceplates
  - increased by around 35 % than the original cost, for a sandwich with S1100 faceplates
- The overall cost of a new design has increased due to the high manufacturing cost of the sandwich panel. On the contrary, the cost of welding in new design has reduced significantly. Approximately 2.5 to 3 times reduction in the cost of welding is observed compared to the original design.

7 Conclusion & Recommendations

7.1 Conclusion

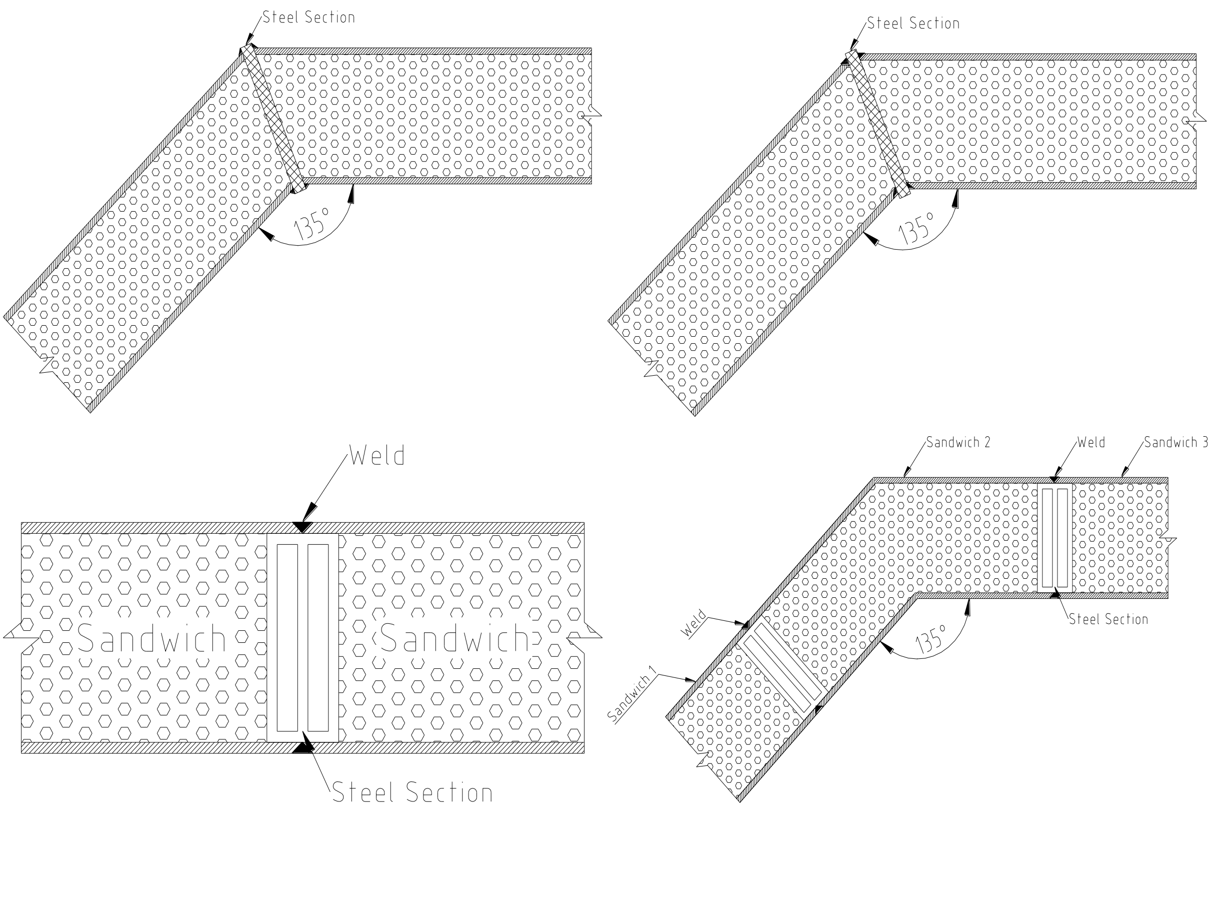
This study concludes that in future it will be possible to use sandwich panels in offshore structures to save a significant amount of weight while taking into account all other considerations. It is not possible to use sandwich panel with faceplates made of S355 steel to replace stiffened plate of S355 steel. Sandwich with extra high strength steel S690 or S1100 can be used for weight reduction. With an improvement in steel strength used for the faceplates, weight reduction margin is improved. In the considered example, with use of sandwich S690 weight reduction of 8 percent is achieved while with sandwich S1100 weight reduction of 20 percent is achieved as compared to original design made from stiffened plate of S355 steel. Unlike this, sandwich S355 results in a 21 percent increase in weight as compared to the original design.

Use of sandwich panels also results in significant cost increment. In cost analysis, it is observed that using the sandwich panel results in cost increase of 30 to 35% as compared to the original cost of the structure. This cost increment is the result of the high manufacturing cost of the sandwich panel. In contrast to this, cost of welding is reduced due to use of the sandwich panel. The welding cost is reduced 2.5 to 3 times that of the original cost. So in terms of costs, it is questioned whether or not using sandwich panels is economically beneficial for offshore structures.

7.2 Recommendations

It is encouraged to continue an investigation for sandwich panels in steel constructions to keep on pushing limitations. Research should be done in investigating connections between sandwich panels. Also, compression and buckling test should be done on a sandwich panel to compare to plate buckling. The lab test should be done on the sandwich panel so as to justify the theory. More research should be done to establish some rule like the Eurocode, which can be referred to during designing a structure with sandwich panels. Continuous improvement and standardization of composite materials and especially standardization of offshore structures may increase economic benefit in future. Research should be done in the field of manufacturing of sandwich panel to achieve cost reduction. A comparison should be done between stiffened plate made from S690 & S1100 and sandwich panel made with faceplates S690 & S1100.

If the sandwich panel is welded in a conventional way then it will result in the melting of foam near the weld area. This will result in the contamination of weld and the improper weld will be created. Therefore, a new type of connection method should be adopted. Some possible types of connection between sandwich panels are shown in the following figures. It might be possible to use friction stir welding. More investigation should be done in this field.



Appendix A Calculation of Buckling Resistance of Sandwich Panel (Column Buckling)

Column buckling theory is used for calculating buckling resistance. For this, method stated in Eurocode is used. Its applicability is unclear and calculation is based on the assumption that core does not contribute to buckling resistance. Equations from euro code are used because they are easy to implement. Also with the help of equations given in Eurocode, it is simple to incorporate global geometric imperfections and local imperfections. With equations in Eurocode, we can see & calculate effects geometric imperfection has on buckling resistance of the sandwich panel. But since its applicability is unknown, comparison is not made between buckling resistance calculated with euro code and failure load calculated with FEM. Theory or sample calculation is only there to show that it can be used for calculating resistance in this specific manner but its applicability and acceptability is unknown.

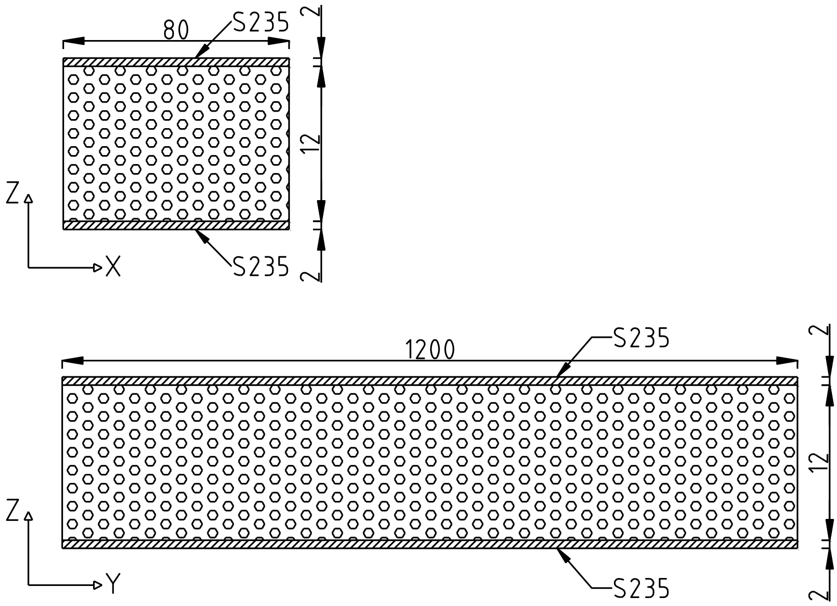


Figure A.1 Sandwich panel 2-12-2

A.1 Analytically

Consider Sandwich Panel used in Havel. The above figure shows the configuration of the sandwich panel.

$l = lenght = 1200\ mm$

$b = widht = 80\ mm$

$t_f = thickness\ of\ face\ plate = 2\ mm$

$t_c = thickness\ of\ core = 12\ mm$

$f_y = yield\ of\ steel = 235\ MPa$

$E_f = modulus\ of\ elsticity\ of\ steel\ (face\ plate) = 210000\ MPa$

$E_c = modulus\ of\ elsticity\ of\ Aluminum\ foam\ (core) = 500\ MPa$

$\upsilon = possion's\ ratio = 0.3$

Moment of Inertia,

$$I = \frac{b(h^3 - t_c^3)}{12} = 15786.67\ mm^4$$

Bending stiffness of sandwich,

$$EI = E\frac{b(h^3 - t_c^3)}{12} = 3.3152\ \times 10^9\ N - mm^2$$

Euler elastic critical buckling load,

$$N_{cr} = \frac{\pi^2 EI}{l^2} = 22698.99\ N$$

Slenderness,

$$\lambda = \sqrt{\frac{Af_y}{N_{cr}}} = \sqrt{\left(\frac{2t_f bf_y}{N_{cr}}\right)} = 1.82$$

Reduction factor,

$$\chi = \frac{1}{\varphi + \sqrt{\varphi^2 - \lambda^2}} = 0.265$$

where,

$$\varphi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2] = 2.33$$

$$\alpha = imperfection\ factor = 0.21$$

Buckling resistance,

$$N_{bRd} = \chi 2t_f bf_y = 19916.69\ N$$

A.2 FEM

Dimensions and geometry of the sandwich panel are taken from chapter 3. Plan i.e. length and width of the sandwich panel is same as that of Havel specimen.

Mesh & Element type:-

For modelling in ANSYS, SOLID186 element type is used. SOLID186 is a second-order 3-D 20-node solid element that exhibits quadratic displacement behaviour. It has 20 nodes having three degrees of freedom per node (x, y and z-direction). SOLID186 is an element which offers the ability to model local bending effects. Because of its quadratic element property, it prevents hour-glassing. It also prevents shear locking.

For materials under pure bending, the shear locking effect is observed. Elements which are exposed to pure bending ideally experience a curved shape change. Linear elements are unable to experience this change. In the case of linear materials, incorrect artificial shear stress is introduced. Because of this shear deformation is generated instead of bending deformation. Overall effect is that under bending moment linear fully integrated components become overly stiff or locked. This shear locking is not observed in the quadratic element due to the introduction of additional nodes.

The computational efficiency is decreased which is an undesirable effect due to the presence of additional nodes. This can be minimized by using a reduced integration solution. Reduced integration may result in an excessively flexible element. This is also known as hourglassing effect. Because of hourglassing, meaningless results are produced. This is due to the fact that normal and shear stresses at point of integration are assumed to be zero. In the through-thickness direction of the panel, if a single layer of elements is used then hourglassing may occur in solid elements of second order. In this study, multiple elements are used in the through-thickness direction so as to prevent hourglassing.

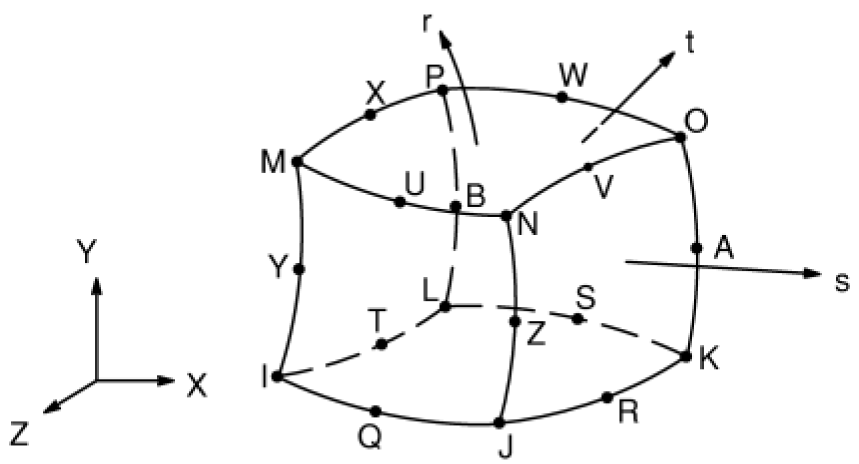


Figure A.2 ANSYS element type SOLID186 [9]

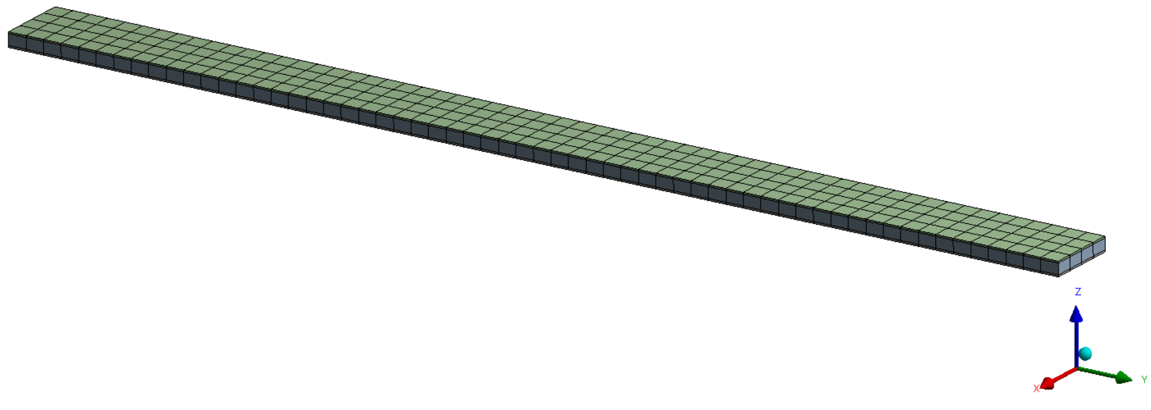


Figure A.3 Sandwich Meshing.

Boundary & Loading Conditions:-

The boundary conditions of the sandwich panel are illustrated in flowing figure. The bottom edge of the sandwich panel is constrained from all degrees of freedom except rotation around x-axis. At top edge rotation around x-axis and displacement along the y-axis is free and all other freedoms are constrained. There are no constraints on the side edges of the sandwich panel. This is so as to stimulate column behaviour of sandwich panel.

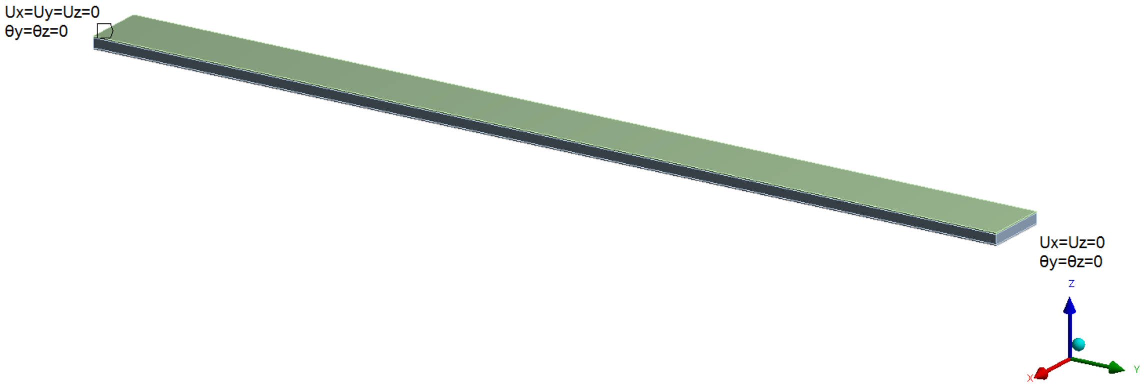


Figure A.4 Boundary conditions of sandwich panel

The sandwich plane is loaded with in-plane compression along the y-axis. Displacement controlled analysis is done. The displacement is applied only on faceplate since the contribution of core to buckling is assumed negligible. The displacement that creates in-plane compression is applied in small increments. The reaction force corresponding to each increment is observed. With an increase in applied displacement corresponding force reaction increases. At a specific point value of force reaction drops (decreases). The force reaction corresponding to this point is considered as a load-carrying capacity of the sandwich panel. The following figure shows applied displacement on the sandwich panel.

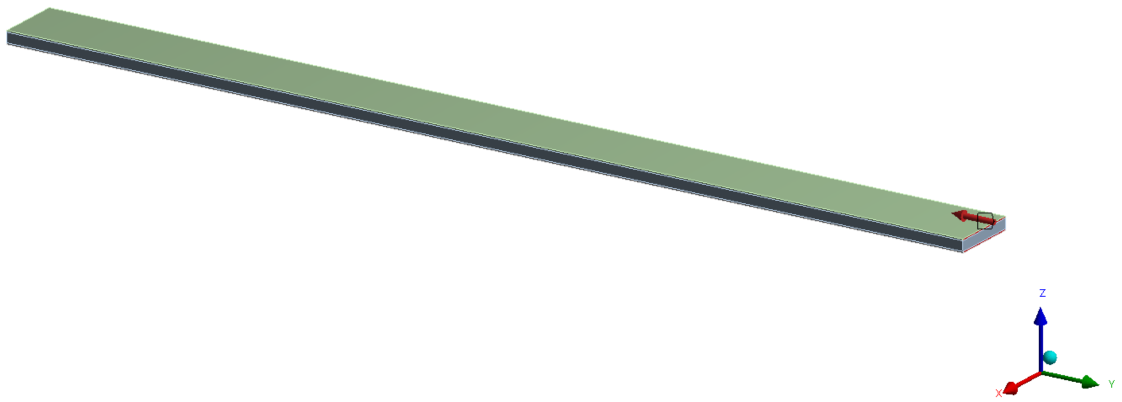


Figure A.5 Loading Condition of sandwich panel

Geometric Imperfections:-

As said previously in chapter 3.3 geometric imperfection seriously affects the strength of steel structure. Strength of member is sensitive to imperfection in shape of its Eigen buckling modes. In the majority of cases, mode shapes based on lowest Eigen buckling mode is sufficient to adequately characterise most influential geometric imperfections and this can be a conservative approach. Therefore, in non-linear analysis mode shape of first and fourth third buckling mode obtained from elastic buckling (Eigen buckling) analysis was used to introduce global and local geometric imperfection shapes respectively.

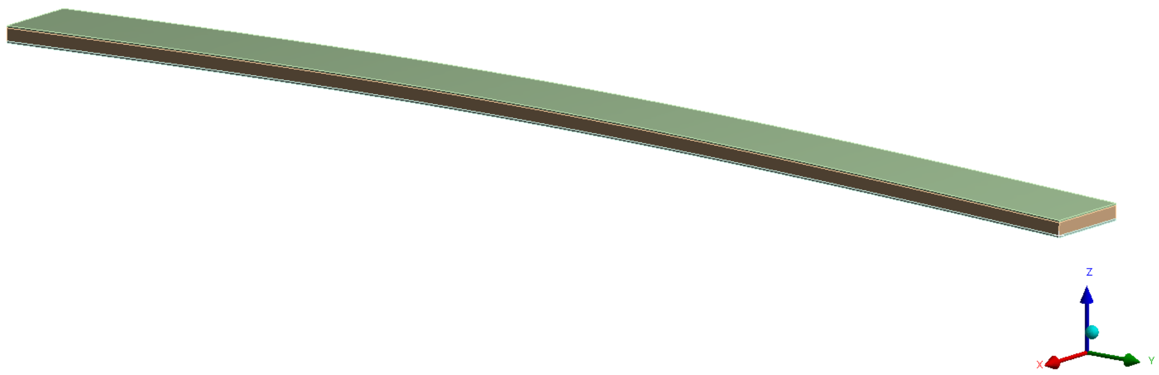


Figure A.6 Model of sandwich panel with global geometric imperfection

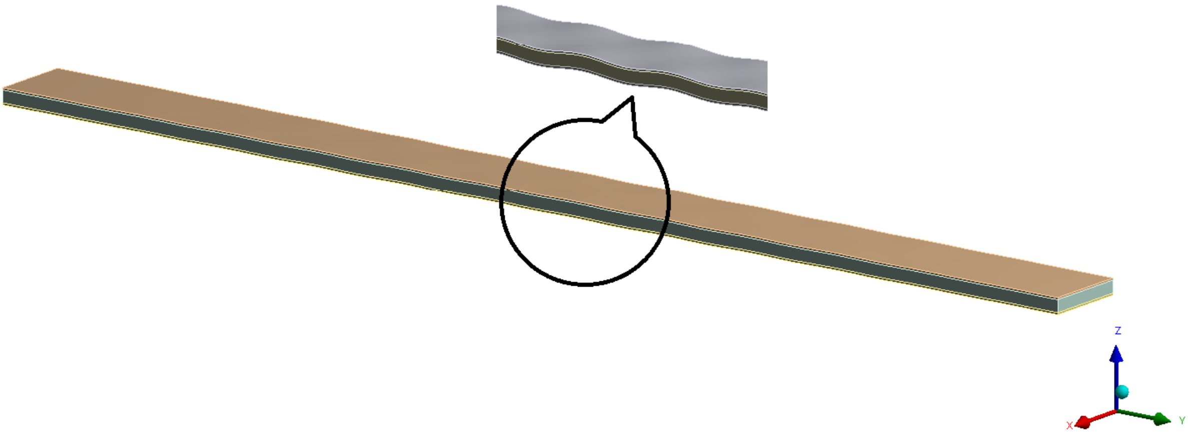


Figure A.7 Model of sandwich panel with local geometric imperfection

Result of FEM:-

Eigenvalue buckling analysis is done on the sandwich panel to find load multiplication factor. Load multiplication factor will give the value of the load at which sandwich will buckle. As per the theory, the first buckling mode is critical in the case of flexural buckling analysis. Generally, in Eigen buckling analysis, it is customary to calculate at least 10 buckling mode but load multiplier value corresponding to the first global buckling mode is only important from point of view of flexural buckling.

The following figure shows the first Eigen buckling mode and load multiplier corresponding to it.

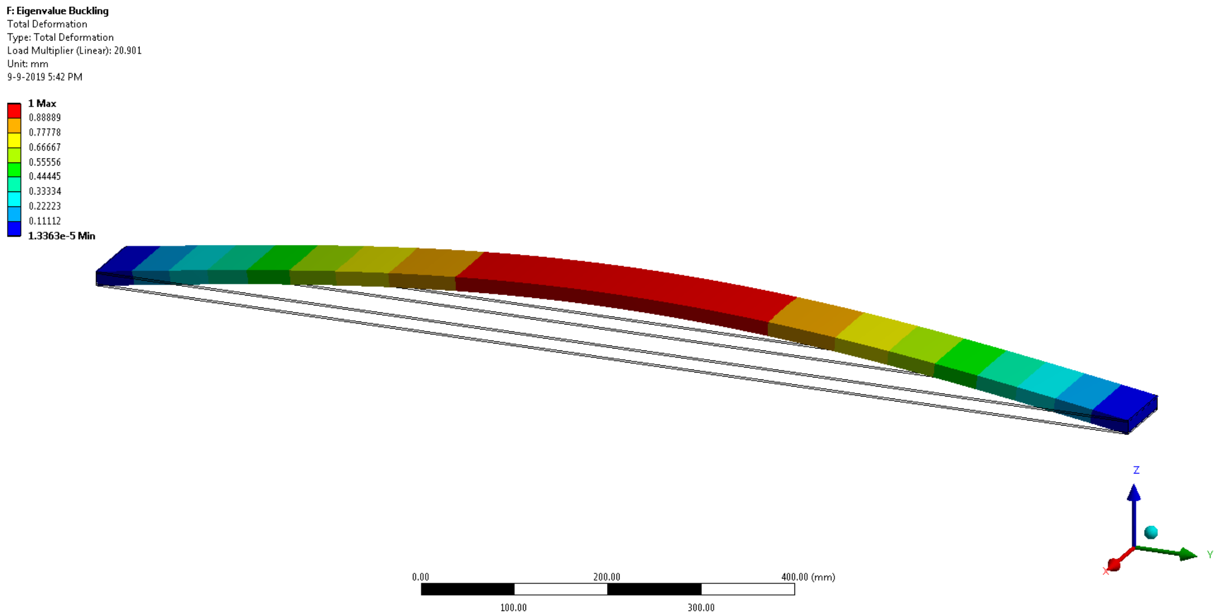


Figure A.8 First Eigen buckling load calculated with ANSYS

$$Load\ Multiplier = 20.901$$

$$Buckling\ load = load\ multiplier * applied\ load$$

$$Buckling\ load = 20.901 * 1000 = 20.901\ KN$$

So buckling load for the sandwich panel is 20.9 Kilo-Newton. Similarly, all other analyses are performed by changing material models and incorporating material imperfection.

Appendix B Calculation of Buckling Resistance of Stiffened Plate (Plate Buckling)

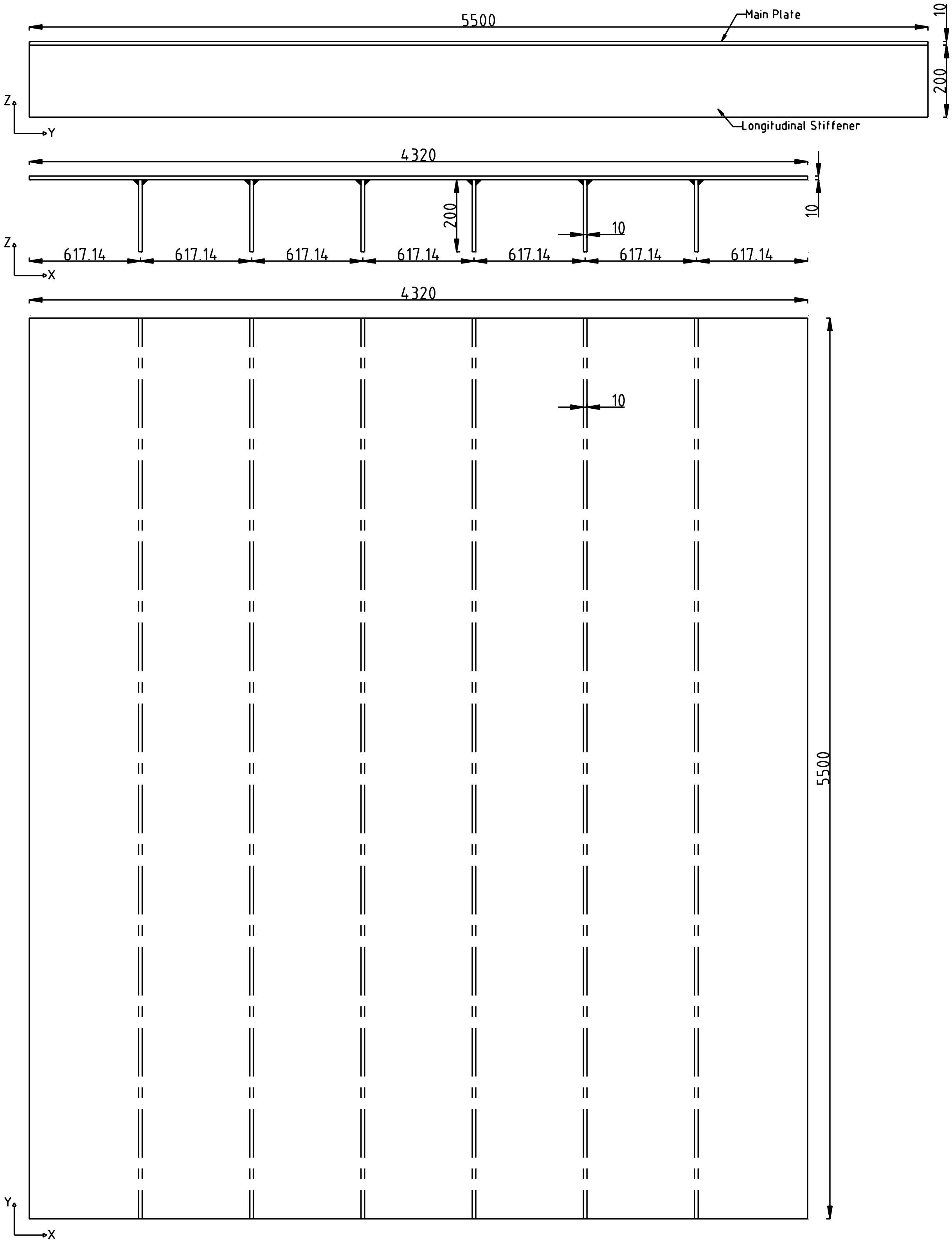


Figure B.1 Stiffened Plate Details

B.1 Analytically

Consider longitudinally stiffened plate as shown below. Buckling strength of plate is calculated according to euro code 1993-1-5

$a = lenght = 5500\ mm$

$b = widht = 4320\ mm$

$fy = yield\ of\ steel = 355\ MPa$

$E = modulus\ of\ elsticity\ of\ steel = 210000\ MPa$

$t_f = thickness\ of\ plate = 10\ mm$

$h_{stiff} = height\ of\ stiffener = 200\ mm$

$t_{stiff} = thickness\ of\ stiffener = 10\ mm$

$n = number\ of\ stiffener = 6$

Plate like buckling,

Second moment of area of the plate,

$$I_p = \frac{bt_f}{12(1-v^2)} = 395604\ mm^4$$

Neutral axis of the whole stiffened plate,

$$z = \frac{6h_{stiff}t_{stiff}\left(\frac{h_{stiff}}{2}\right) + bt_f\left(h_{stiff} + \frac{t_f}{2}\right)}{6h_{stiff}t_{stiff} + bt_f} = 182.17\ mm$$

Second moment of area of the whole stiffened plate,

$$I_{sl} = n\left[\frac{t_{stiff}h_{stiff}^3}{12} + t_{stiff}h_{stiff}\left(z - \frac{h_{stiff}}{2}\right)^2\right] + \left[\frac{bt_f^3}{12} + bt_f\left(h_{stiff} + \frac{t_f}{2} - z\right)^2\right] = 1.439 * 10^8\ mm^4$$

Sum of the gross area of individual longitudinal stiffeners,

$$A_{sl} = nh_{stiff}t_{stiff} = 12000\ mm^2$$

Gross area of the plate,

$$A_p = bt_f = 43200\ mm^2$$

$$\alpha = \frac{a}{b} = 1.273$$

$$\delta = \frac{A_{sl}}{A_p} = 0.278$$

$$\gamma = \frac{I_{sl}}{I_p} = 363.75$$

Since $\alpha \leq \sqrt[4]{\gamma}$ and $\Psi = 1$ pure compression,

$$k_{\sigma,p} = \frac{2[(1+\alpha^2)^2 + \gamma - 1]}{\alpha^2(1+\Psi)(1+\delta)} = 178.46$$

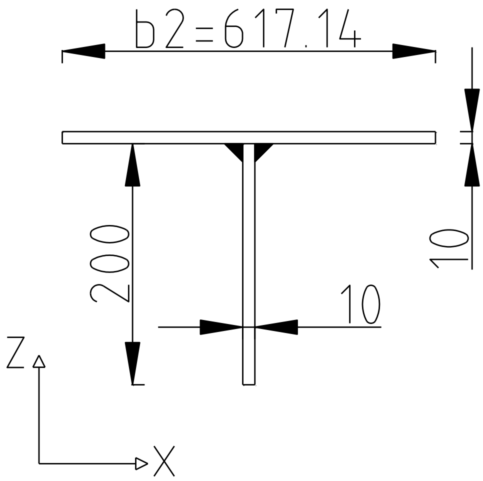
$$\sigma_E = \frac{\pi^2 E t_f^2}{12(1-\upsilon^2)b^2} = 1.016\ MPa$$

Elastic critical plate buckling stress,

$$\sigma_{cr,p,\text{analy}} = k_{\sigma,p}\sigma_E = 181.31\ MPa$$

Stiffened plate can also behave as per column like buckling so it should be checked.

Column like buckling,



Neutral axis,

$$z = \frac{h_{stiff}t_{stiff}\left(\frac{h_{stiff}}{2}\right) + b1\ t_f\left(h_{stiff} + \frac{t_f}{2}\right)}{h_{stiff}t_{stiff} + b1\ t_f} = 179.3\ mm$$

Second moment of area of the gross cross-section of the stiffener and the adjacent parts of the plate, relative to the out of plane bending of the plate,

$$I_{sl,1} = \left[\frac{t_{stiff}h_{stiff}^3}{12} + t_{stiff}h_{stiff}\left(z - \frac{h_{stiff}}{2}\right)^2\right] + \left[\frac{b1\ t_f^3}{12} + b1\ t_f\left(h_{stiff} + \frac{t_f}{2} - z\right)^2\right] = 23.37 * 10^6\ mm^4$$

The gross cross-sectional area of the stiffener and the adjacent parts of the plate,

$$A_{sl,1} = h_{stiff}t_{stiff} + b1\ t_f = 8171.4\ mm^2$$

The elastic critical column buckling stress of the stiffener closest to the panel edge with the highest compressive stress,

$$\sigma_{cr,sl} = \frac{\pi^2 E I_{sl,1}}{A_{sl,1}a^2} = 195.75\ MPa$$

Since there is pure compression, elastic critical buckling stress is,

$$\sigma_{cr,c} = \sigma_{cr,sl} = 195.75\ MPa$$

Interaction between column-like & plate-like buckling,

$$\xi = \frac{\sigma_{cr,p}}{\sigma_{cr,c}} - 1 = \frac{181.31}{195.75} - 1 = -0.071$$

But $0 \leq \xi \leq 1$. Therefore, $\xi = 0$

$$\rho_c = (\rho - \chi_c)\xi(2 - \xi) + \chi_c$$

$$\rho_c = \chi_c$$

Where, χ_c is the reduction factor due to column buckling

ρ is the reduction factor due to plate buckling

Therefore, the stiffened plate will show a column-like buckling behaviour.

In this thesis, plate-like buckling of the stiffened plate is considered & resistance is calculated accordingly.

Effective width of internal compression flange,

$$b_2 = \frac{b}{n+1} = 617.14\, mm$$

$$c_2 = \left(\frac{b}{n+1} - t_{stiff}\right) = 607.14\, mm$$

$$\Psi = 1$$

$$k_\sigma = 4$$

$$\lambda_{p2} = \sqrt{\frac{f_y}{\left(k_\sigma \frac{\pi^2 E}{12(1-\upsilon^2)\left(\frac{c_2}{t_f}\right)^2}\right)}} = 1.313$$

$$\rho = \frac{\lambda_{p2} - 0.055(3+\Psi)}{\lambda_{p2}^2} = 0.633$$

$$b_{2eff} = \rho c_2 = 384.8\, mm$$

Effective width of external compression flange,

$$b_1 = \frac{b}{n+1} = 617.14\, mm$$

$$c_1 = \left(\frac{b}{n+1} - \frac{t_{stiff}}{2}\right) = 612.14\, mm$$

$$\Psi = 1$$

$$k_\sigma = 4$$

$$\lambda_{p1} = \sqrt{\frac{f_y}{\left(k_\sigma \frac{\pi^2 E}{12(1-\upsilon^2)\left(\frac{c_1}{t_f}\right)^2}\right)}} = 1.32$$

$$\rho = \frac{\lambda_{p1} - 0.188}{\lambda_{p1}^2} = 0.62$$

$$b_{1eff} = \rho c_1 = 385\ mm$$

Effective cross-section of the stiffened plate,

$$A_{c,eff,loc} = \left(\frac{b_{1eff}}{2}2 + \frac{b_{2eff}}{2}10\right)tf + h_{stiff}t_{stiff}n = 35094\ mm^2$$

Cross-section of the stiffened plate,

$$A_c = \left(b - \frac{b_1}{2}2\right)tf + h_{stiff}t_{stiff}n = 49029\ mm^2$$

Ratio of area,

$$\beta_{AC} = \frac{A_{c,eff,loc}}{A_c} = 0.7$$

Relative plate slenderness of equivalent plate is,

$$\lambda_p = \sqrt{\frac{\beta_{AC} f_y}{\sigma_{cr,p,\mathrm{analy}}}} = 1.1$$

Reduction factor as per winter formula,

$$\rho = \frac{1}{\lambda_p}\left(1 - \frac{0.22}{\lambda_p}\right) = 0.7$$

Buckling Resistance,

$$N_{bRd} = \rho A_c f_y = 12183.7\ KN$$

B.2 FEM

Dimensions and geometry of the stiffened plate are taken from Huisman structure chapter 2.8.1. Length, width and thickness of the base plate and that of stiffeners are exactly same as a stiffened plate used in Huisman structure. The stiffened plate is modelled in ANSYS and results are analysed.

Mesh & Element type:-

For modelling in ANSYS, SOLID186 element type is used. SOLID186 is a second-order 3-D 20-node solid element that exhibits quadratic displacement behaviour. It has 20 nodes having three degrees of freedom per node (x, y and z-direction). SOLID186 is an element which offers the ability to model local bending effects. Because of its quadratic element property, it prevents hour-glassing. It also prevents shear locking.

For materials under pure bending, the shear locking effect is observed. Elements which are exposed to pure bending ideally experience a curved shape change. Linear elements are unable to experience this change. In the case of linear materials, incorrect artificial shear stress is introduced. Because of this shear deformation is generated instead of bending deformation. Overall effect is that under bending moment linear fully integrated components become overly stiff or locked. This shear locking is not observed in the quadratic element due to the introduction of additional nodes.

The computational efficiency is decreased which is an undesirable effect due to the presence of additional nodes. This can be minimized by using a reduced integration solution. Reduced integration may result in an excessively flexible element. This is also known as hourglassing effect. Because of hourglassing, meaningless results are produced. This is due to the fact that normal and shear stresses at point of integration are assumed to be zero. In the through-thickness direction of the panel, if a single layer of elements is used then hourglassing may occur in solid elements of second order. In this study, multiple elements are used in the through-thickness direction so as to prevent hourglassing.

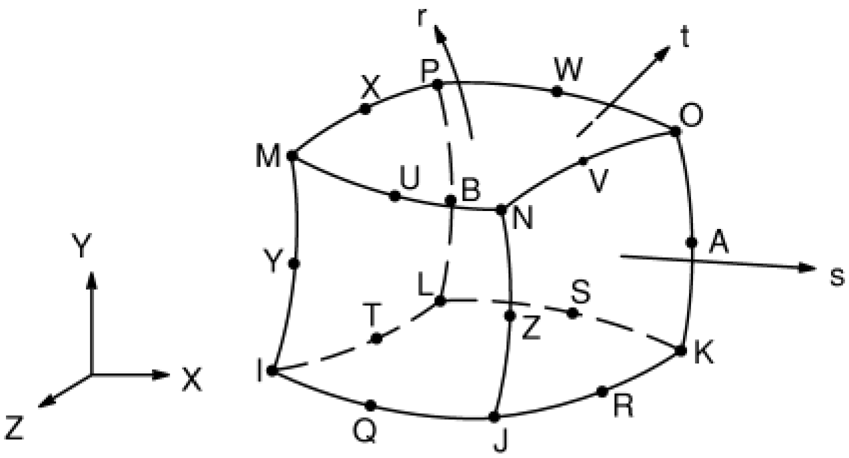


Figure B.2 ANSYS element type SOLID186 [9]

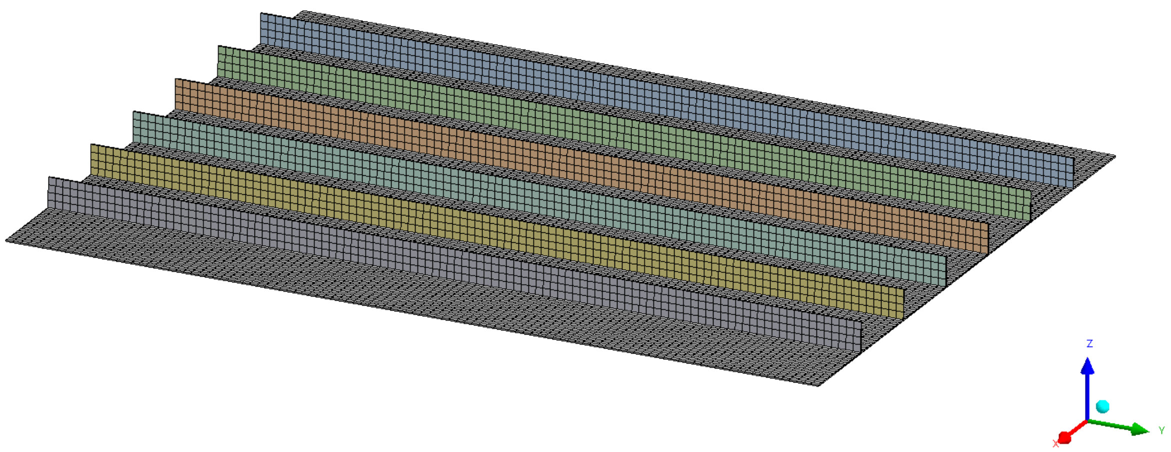


Figure B.3 Sandwich Meshing

Boundary & Loading Conditions:-

The boundary conditions of the stiffened plate are illustrated in flowing figure. The bottom edge of the sandwich panel is constrained from all degrees of freedom except rotation around x-axis. Whereas for top edge rotation around x-axis and displacement along the y-axis is free and all other freedoms are constrained. Displacement of side edges in the direction perpendicular to the plane is restricted i.e. displacement in the z-direction is restrained.

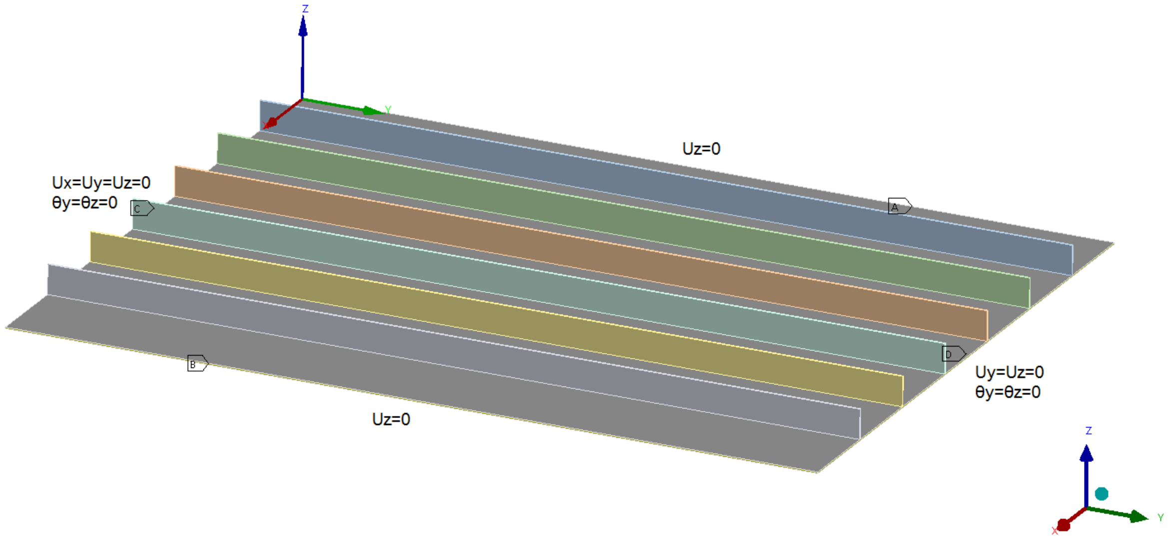


Figure B.4 Boundary conditions of sandwich panel

The stiffened plate is loaded with in-plane compression along the y-axis. The load is applied on the cross-section of the stiffened plate. Load of 1000 Newton is applied. The following figure shows loading on the stiffened plate.

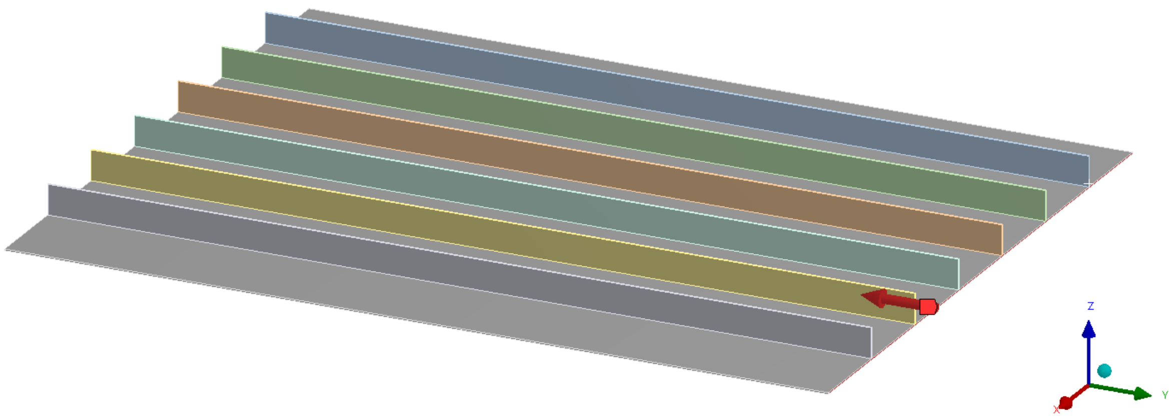
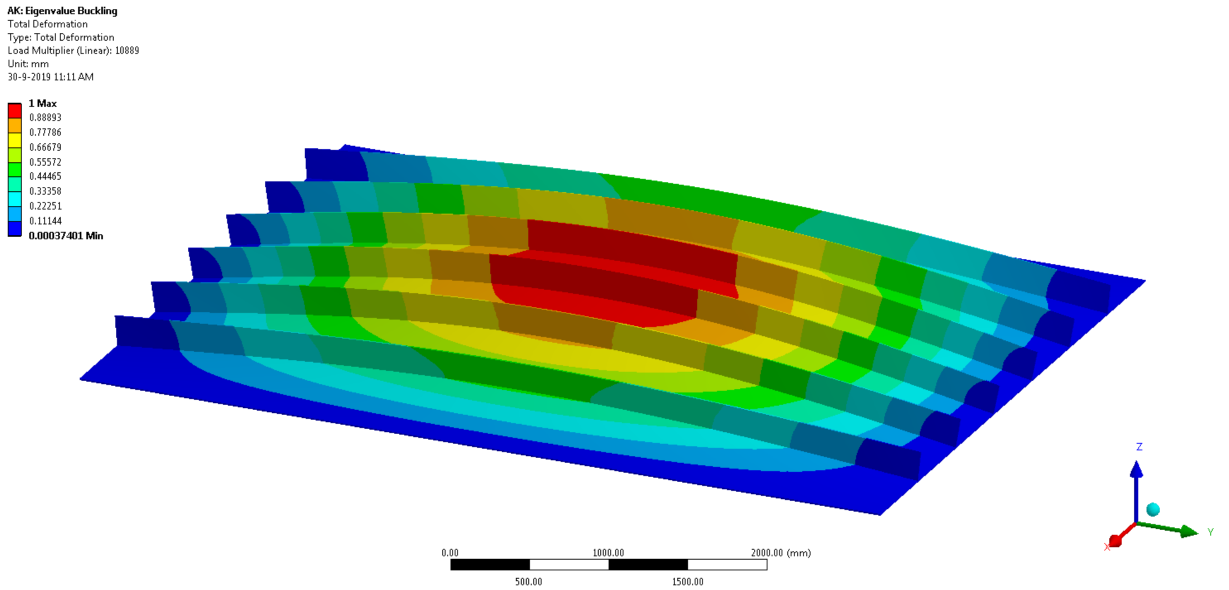


Figure B.5 Loading Condition of sandwich panel

Result of FEM:-

Eigenvalue buckling analysis is done on the stiffened plate to find load multiplication factor. Load multiplication factor will give a value of the load at which the stiffened plate will buckle. As per the theory, the first buckling mode is critical in the case of flexural buckling analysis. Generally, in Eigen buckling analysis, it is customary to calculate at least 10 buckling mode but load multiplier value corresponding to the first mode is only important from point of view of flexural buckling. Also, it is important to find out the value of stress at the buckling load. For this minimum principal stress is calculated.

The following figure shows the first Eigen buckling mode and load multiplier corresponding to it.



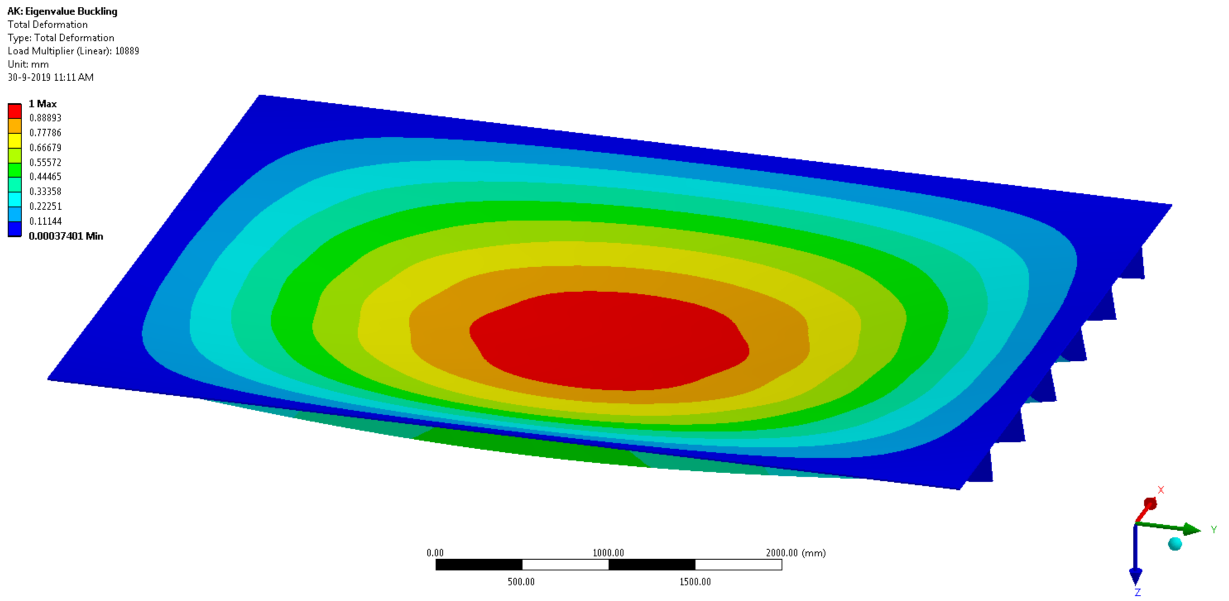


Figure B.6 First Eigen buckling load calculated with ANSYS

$$Load\ Multiplier = 10889$$

$$Buckling\ load = load\ multiplier * applied\ load$$

$$Buckling\ load = 10889 * 1000 = 10889\ KN$$

Now, from the above figure position of maximum displacement can be observed. After this, minimum principal stress at the same position will be measured. In practice it is difficult to exactly point out this position, therefore some points near the point of maximum deformation are arbitrarily chosen and, minimum principal stress at all these points will be measured. The following figure will give an idea of this.

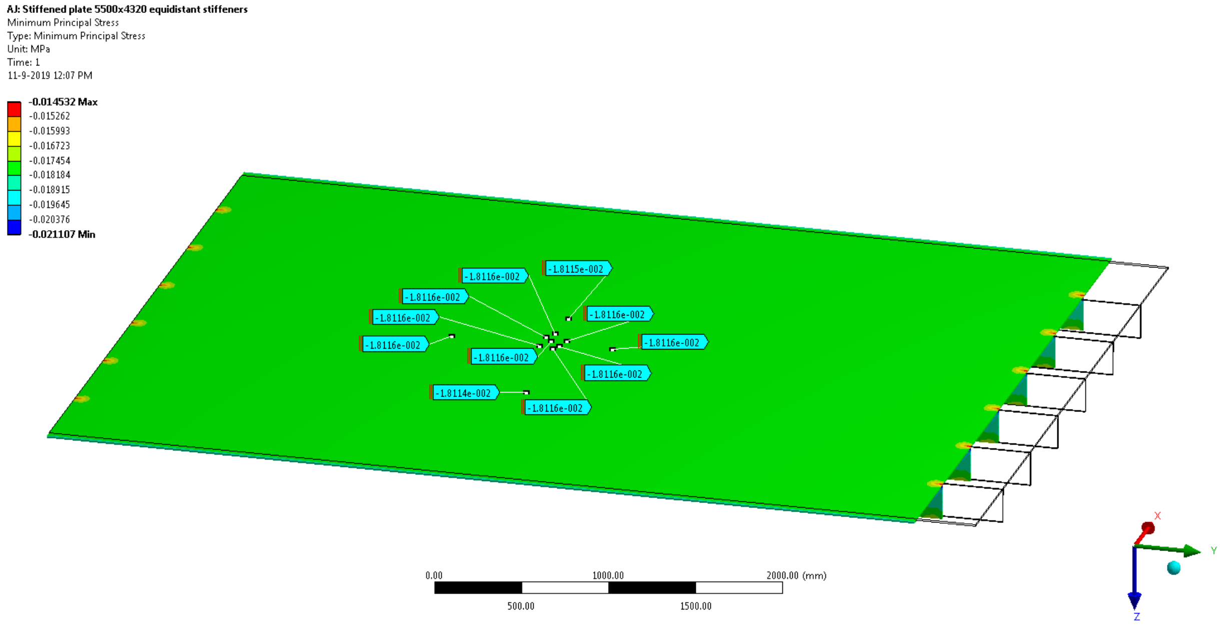


Figure B.7 Minimum principle stress calculated with ANSYS

It can be observed that faces are in uniform compression so the values of minimum principal stress at all points are equal. If values at different points would have been different then it is better to take an average. But in this case, the average will be exactly the same as the value at an individual point.

$$minimum\ principle\ stress = 1.8116 * 10^{-2}\ MPa$$

Form these two value i.e. load multiplier and minimum principal stress, stress at buckling can be calculated as follows,

$$stress = load\ multiplier * minimum\ principle\ stress$$

$$stress = 10889 * 1.8116 * 10^{-2}$$

$$stress = 197.26\ MPa$$

Another simple, approximate method that can be applied in this case, to calculate stress at buckling would be, dividing buckling load with cross-section area on which load is applied.

$$\sigma_{cr,p,FEM} = \frac{F_{FEM}}{A} = \frac{F_{FEM}}{bt_p + nt_{stiff}h_{stiff}} = \frac{10889 * 10^3}{4320 * 10 + 6 * 200 * 10} = 197.26\ MPa$$

To find the resistance of sandwich panel non-linear analysis of sandwich panel is performed. In this non-linear analysis rather than applying the load, displacement is applied and the resultant force is calculated. The displacement is applied in short increment until the point where it is not possible to achieve force convergence and model fails. The displacement increment & force reaction corresponding to it is arranged in tabular form. The maximum force reaction can be treated as the load-carrying capacity of the sandwich panel. In addition, to visualise this, graph between displacement increment and corresponding force reaction is plotted as follows,

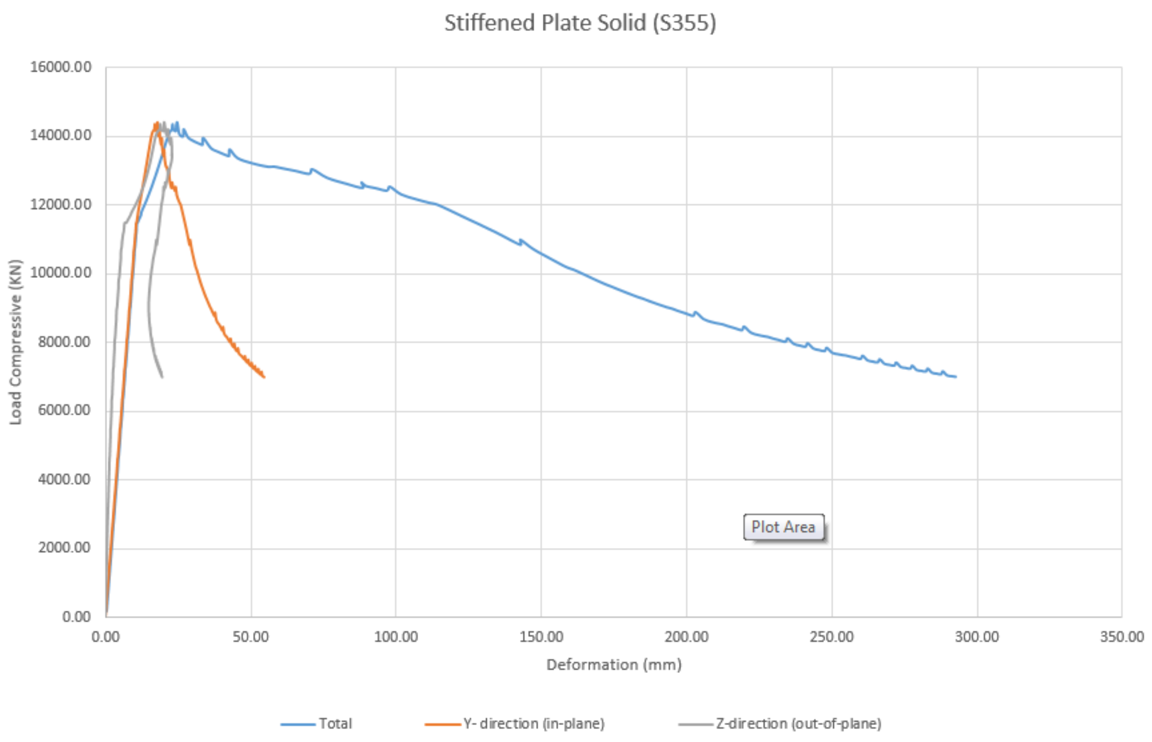


Figure B.8 Nonlinear Buckling Analysis of Stiffened Plate S355

From the Ansys nonlinear analysis of stiffened plate resistance comes out to be 14371 KN. This can be observed from the above graph.

Appendix C Calculation of Weight of Stiffened Plate

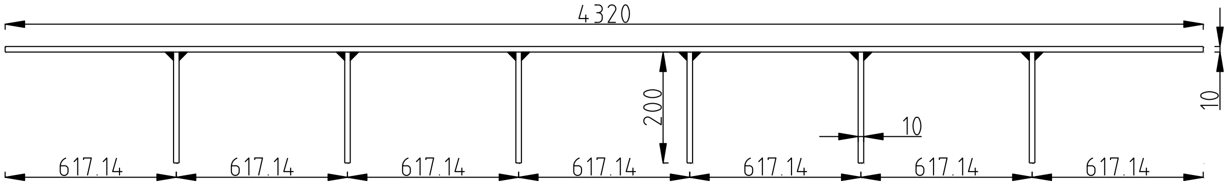


Figure C.1 Cross-section of stiffened plate

Above figure shows, stiffened plate considered for example. The stiffened plate consists of the base plate having the length of 5500 mm, the width of 4320 mm and thickness of 10 mm. The plate is stiffened with six stiffeners of the same length, 200 mm height and 10 mm thickness. The stiffened plate is made of steel S355.

$a = lenght = 5500\ mm$

$b = widht = 4320\ mm$

$t_f = thickness\ of\ plate = 10\ mm$

$h_{stiff} = height\ of\ stiffener = 200\ mm$

$t_{stiff} = thickness\ of\ stiffener = 10\ mm$

$n = number\ of\ stiffener = 6$

$density\ of\ steel = 7.85 \times\ 10^{-3}\ gram/mm^3$

Weight of base plate,

$$w_{main\ plate} = bt_f 7.85\ \times 10^{-3} = 339.12\ gram/mm$$

Weight of each stiffener,

$$w_{stiffener} = h_{stiff} t_{stiff} 7.85 \times 10^{-3} = 15.7\ gram/mm$$

Weight of stiffened plate per unit length,

$$w_{stiffened\ plate} = w_{main\ plate} + 6 \times w_{stiffener} = 433.32\ gram/mm$$

Weight of stiffened plate,

$$W_{stiffened\ plate} = a \times w = 2383.26\ KG$$

Appendix D Calculation of Buckling Resistance of Sandwich Panel (Plate Buckling)

Sandwich with faceplates S690

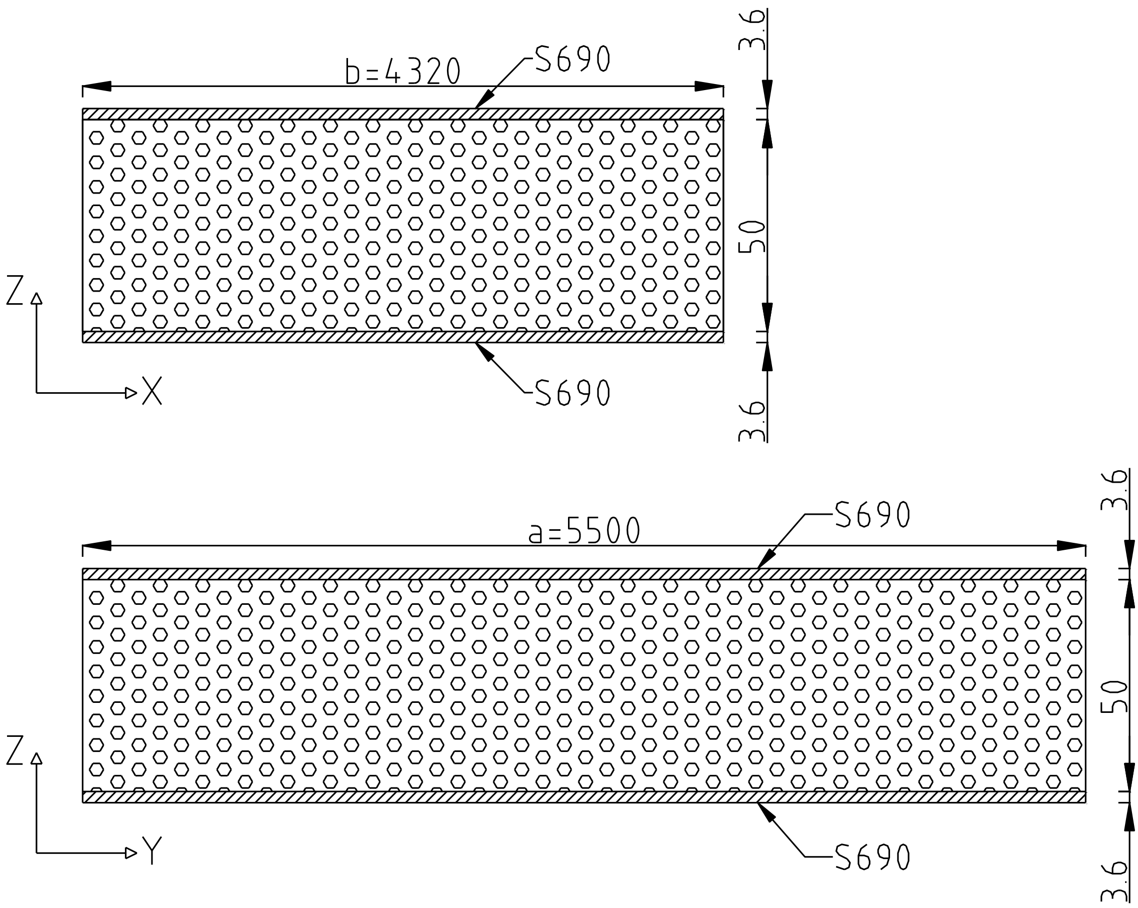


Figure D.1 Sandwich panel 3-70-3

D.1 Analytically

Consider longitudinal sandwich panel as shown below. Buckling strength of the plate is calculated as follows,

$a = lenght = 5500\ mm$

$b = widht = 4320\ mm$

$f_y = yield\ of\ steel = 690\ MPa$

$E = modulus\ of\ elsticity\ of\ steel = 210000\ MPa$

$t_f = thickness\ of\ face\ plate = 3.6\ mm$

$t_c = thickness\ of\ core = 50\ mm$

$\upsilon = possion's\ ratio = 0.3$

$n = number\ of half sine\ in\ transverse\ direction$

$m = number\ of half sine\ in\ logitudinal\ direction$

Flexural stiffness for an isotropic sandwich panel,

$$D_1 = D_2 = D_3 = \frac{E_f t_c^2 t_f}{2(1-\upsilon^2)} = 1.04 * 10^9$$

Critical buckling load per unit width can be,

$$N_{cr} = -\frac{\pi^2 a^2}{m^2}\left[D_1\left(\frac{m}{a}\right)^4 + 2D_3\left(\frac{m}{a}\right)^2\left(\frac{n}{b}\right)^2 + D_2\left(\frac{n}{b}\right)^4\right] = 2325.02\ N/mm$$

Critical stress,

$$\sigma_{cr,sandwich,analy} = \frac{N_{cr}}{2t_f} = 332.92\ MPa$$

$$\lambda_p = \sqrt{\frac{f_y}{\sigma_{cr,sandwich,analy}}} = 1.46$$

As per winter formula,

$$\rho = \frac{1}{\lambda_p}\left(1 - \frac{0.22}{\lambda_p}\right) = 0.58$$

Buckling Resistance,

$$N_{bRd} = 2t_f b f_y \rho = 12472.5\ KN$$

D.2 FEM

Dimensions and geometry of the sandwich panel are taken from chapter 4. Plan i.e. length and width of the sandwich panel is same as that of the stiffened plate. The only varying parameter is thickness, which comprises of thickness of the faceplates and thickness of the core.

Mesh & Element type:-

For modelling in ANSYS, SOLID186 element type is used. SOLID186 is a second-order 3-D 20-node solid element that exhibits quadratic displacement behaviour. It has 20 nodes having three degrees of freedom per node (x, y and z-direction). SOLID186 is an element which offers the ability to model local bending effects. Because of its quadratic element property, it prevents hour-glassing. It also prevents shear locking.

For materials under pure bending, the shear locking effect is observed. Elements which are exposed to pure bending ideally experience a curved shape change. Linear elements are unable to experience this change. In the case of linear materials, incorrect artificial shear stress is introduced. Because of this shear deformation is generated instead of bending deformation. Overall effect is that under bending moment linear fully integrated components become overly stiff or locked. This shear locking is not observed in the quadratic element due to the introduction of additional nodes.

The computational efficiency is decreased which is an undesirable effect due to the presence of additional nodes. This can be minimized by using a reduced integration solution. Reduced integration may result in an excessively flexible element. This is also known as hourglassing effect. Because of hourglassing, meaningless results are produced. This is due to the fact that normal and shear stresses at point of integration are assumed to be zero. In the through-thickness direction of the panel, if a single layer of elements is used then hourglassing may occur in solid elements of second order. In this study, multiple elements are used in the through-thickness direction so as to prevent hourglassing.

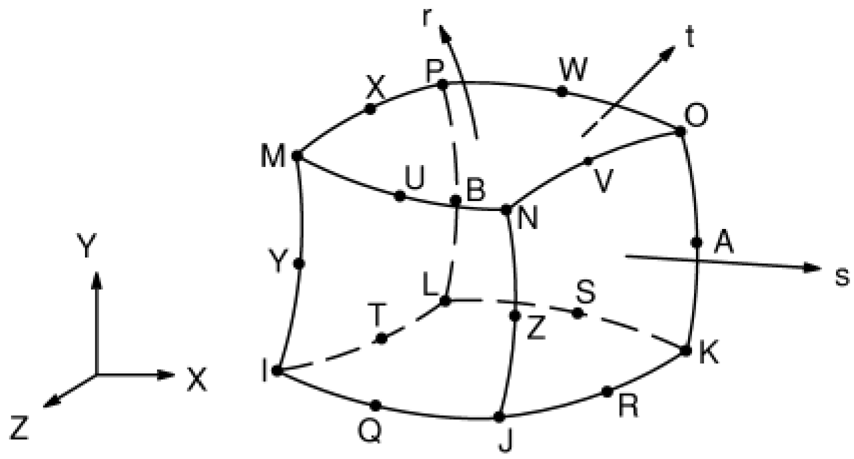


Figure D.2 ANSYS element type SOLID186 [9]

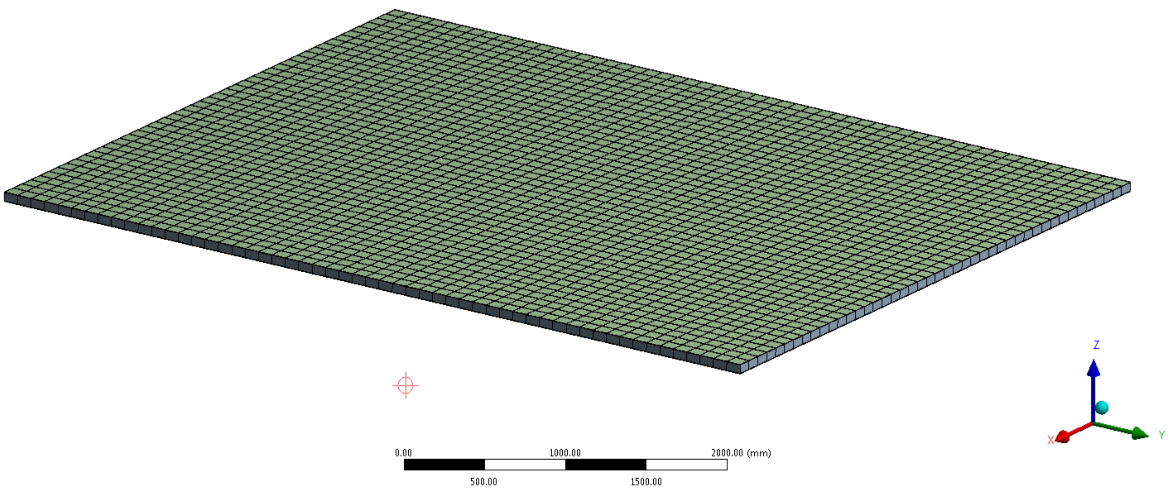


Figure D.3 Sandwich Meshing.

Boundary & Loading Conditions:-

The boundary condition of the sandwich panel is illustrated in flowing figure. The bottom edge of the sandwich panel is constrained from all degrees of freedom except rotation around x-axis. Whereas at top edge rotation around x-axis and displacement along the y-axis is, free and all other freedoms are constrained. Displacement of side edges in the direction perpendicular to the plane is restricted i.e. displacement in the z-direction is restrained.

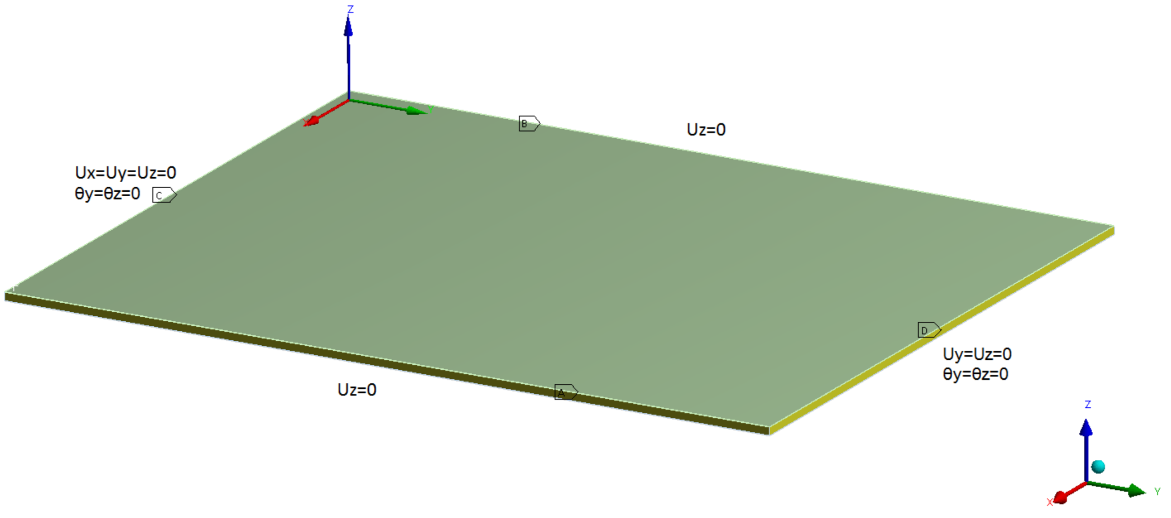


Figure D.4 Boundary conditions of sandwich panel

The sandwich plane is loaded with in-plane compression along the y-axis. The load is applied only on faceplate since the contribution of core to buckling is assumed negligible. Load of 1000 Newton is applied. The following figure shows loading on the sandwich panel.

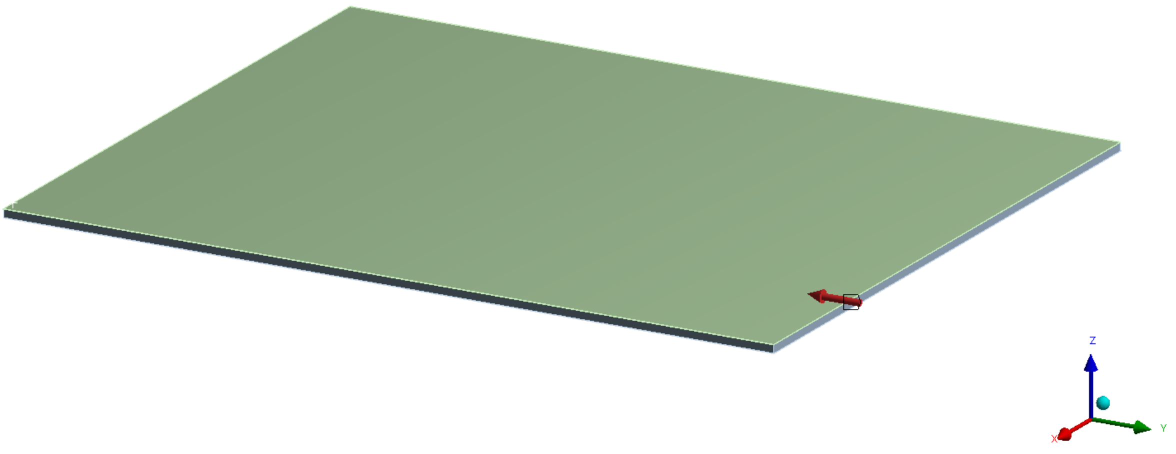


Figure D.5 Loading Condition of sandwich panel

Result of FEM:-

Eigenvalue buckling analysis is done on the sandwich panel to find load multiplication factor. Load multiplication factor will give a value of the load at which sandwich will buckle. As per the theory, the first buckling mode is critical in the case of flexural buckling analysis. Generally, in Eigen buckling analysis, it is customary to calculate at least 10 buckling mode but load multiplier value corresponding to the first mode is only important from point of view of flexural buckling. Also, it is important to find out the value of stress at the buckling load. For this minimum principal, stress is calculated. The following figure shows the first Eigen buckling mode and load multiplier corresponding to it.

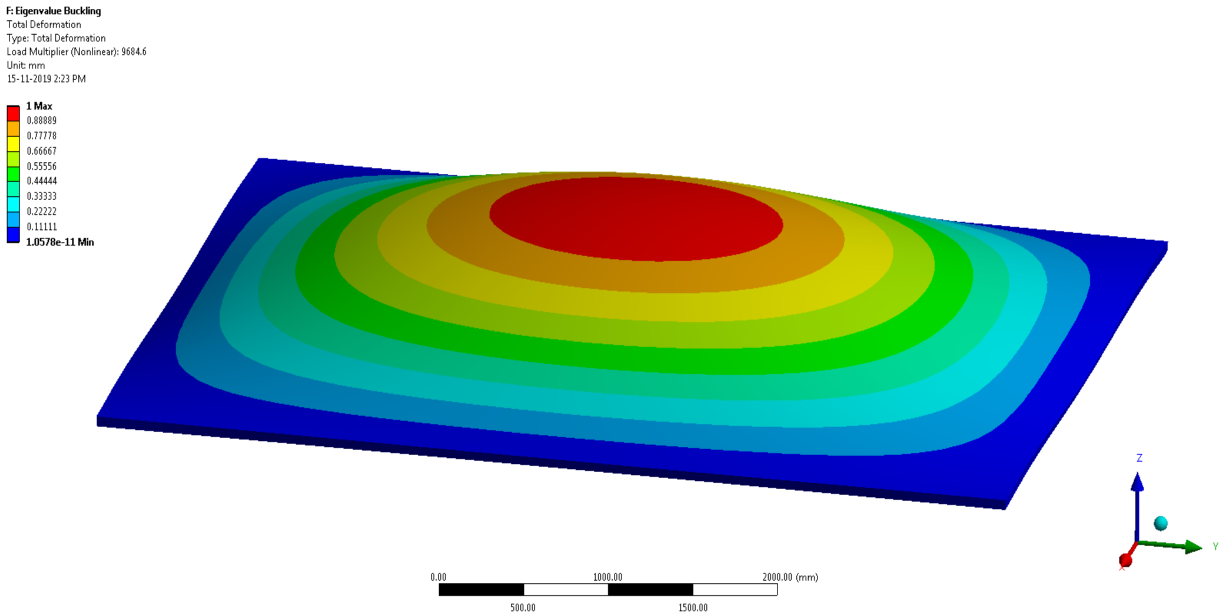


Figure D.6 First Eigen buckling load calculated with ANSYS

$$Load\ Multiplier = 9684.6$$
$$Buckling\ load = load\ multiplier * applied\ load$$
$$Buckling\ load = 9684.6 * 1000 = 9684.6\ KN$$

Now, from the above figure position of maximum displacement can be observed. After this, minimum principal stress at the same position will be measured but since it is difficult to exactly point out this position, some points are arbitrarily chosen near the point of maximum deformation and, minimum principal stress at all these points will be measured. The following figure will give an idea of this.

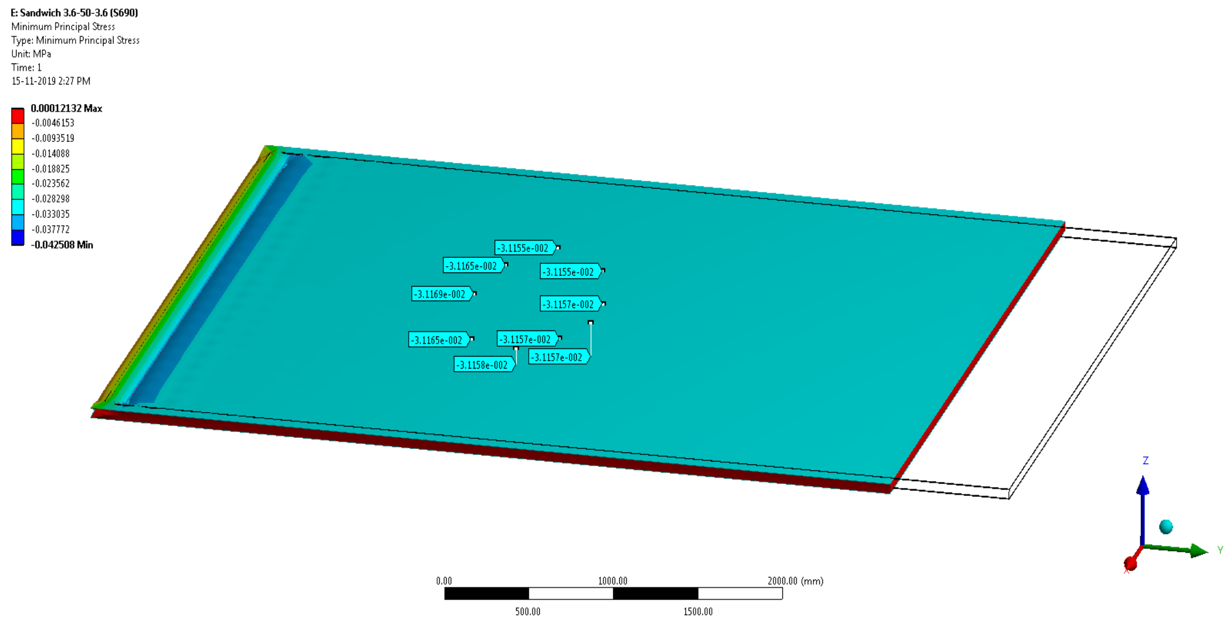


Figure D.7 Minimum principle stress calculated with ANSYS

It can be observed that faces are in uniform compression so the values of minimum principal stress at all points are equal. If values at different points would have been different then it is better to take an average. But in this case, the average will be exactly the same as the value at an individual point.

$$minimum\ principle\ stress = 3.11 * 10^{-2}\ MPa$$

Form these two value i.e. load multiplier and minimum principal stress, stress at buckling can be calculated as follows,

$$stress = load\ multiplier * minimum\ principle\ stress$$
$$stress = 9684.6 * 3.11 * 10^{-2}$$
$$stress = 301.19\ MPa$$

Another simple method that can be applied in this case, to calculate stress at buckling, would be dividing buckling load with cross-section area on which load is applied.

$$\sigma_{cr,Sandwich,FEM} = \frac{F_{FEM}}{A} = \frac{F_{FEM}}{2bt_f} = \frac{9684.6 * 10^3}{2 * 3.6 * 4320} = 311.36\ MPa$$

To find the resistance of sandwich panel non-linear analysis of sandwich panel is performed. In this non-linear analysis rather than applying the load, displacement is applied and the resultant force is calculated. The displacement is applied in short increment until the point where it is not possible to achieve force convergence and model fails. The displacement increment & force reaction corresponding to it is arranged in tabular form. The maximum force reaction can be treated as a load-carrying capacity of the sandwich panel. In addition, to visualise this, graph between displacement increment and corresponding force reaction is plotted as follows,

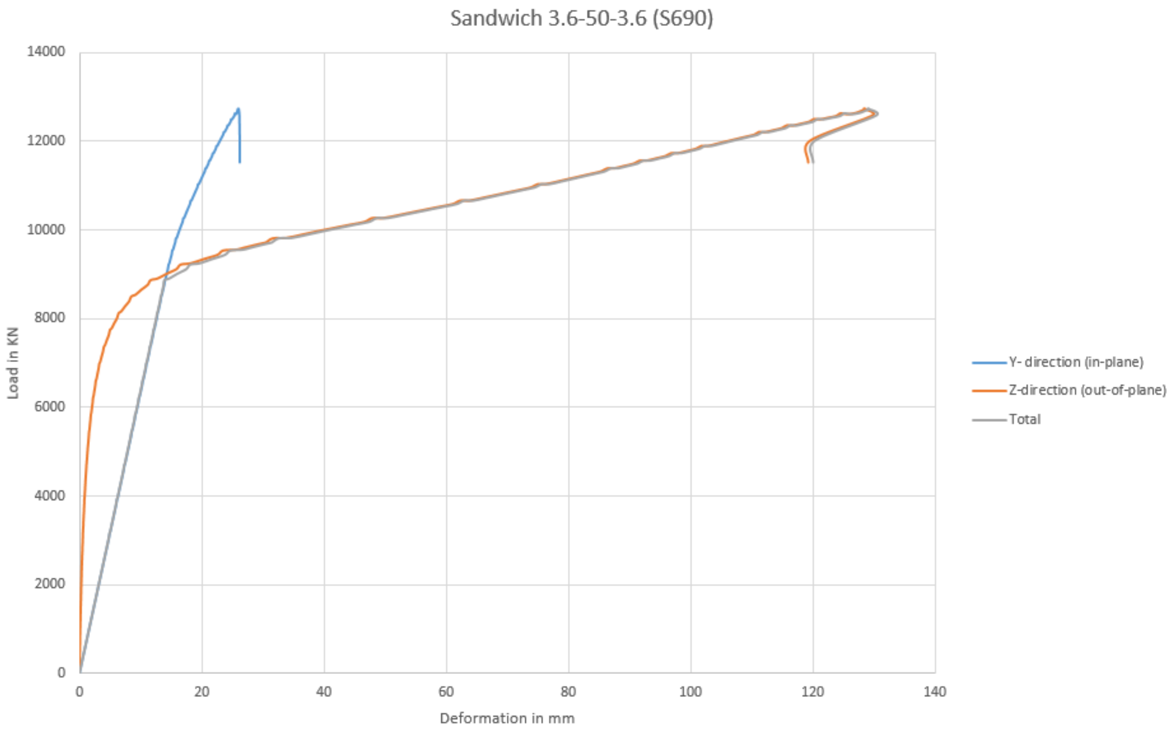
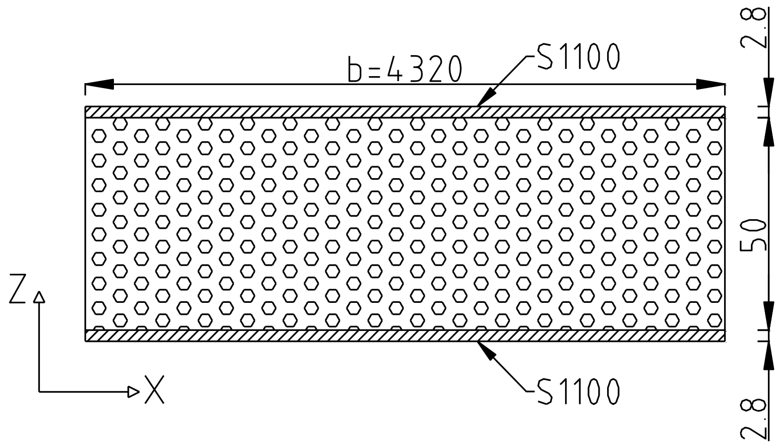


Figure D.8 Nonlinear Buckling Analysis of Sandwich S690

From the Ansys nonlinear analysis of sandwich S690, the load-carrying capacity of the sandwich panel comes out to be 12738 KN. This can be observed in the above graph.

Sandwich with faceplates S1100



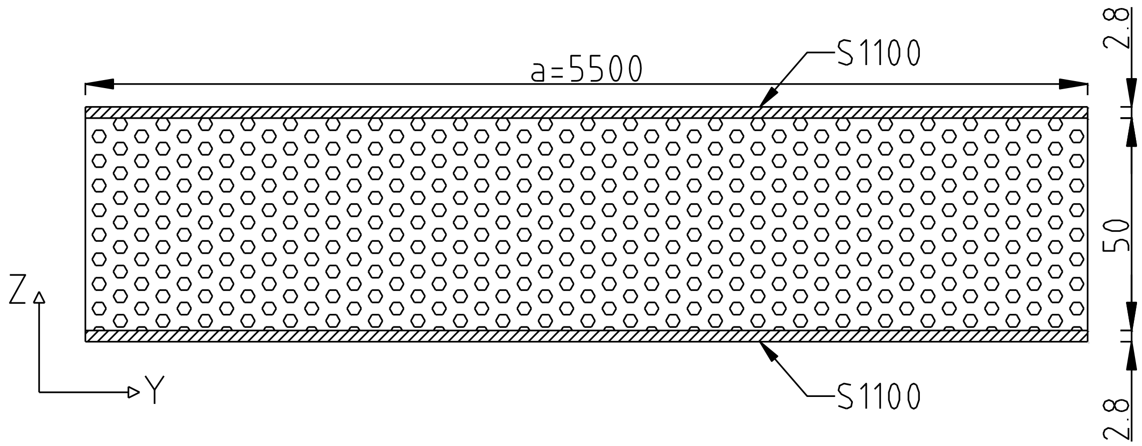


Figure D.9 Sandwich panel 2.8-50-2.8

D.3 Analytically

Consider longitudinal sandwich panel as shown below. Buckling strength of the plate is calculated as follows,

$a = lenght = 5500\ mm$

$b = widht = 4320\ mm$

$f_y = yield\ of\ steel = 1100\ MPa$

$E = modulus\ of\ elsticity\ of\ steel = 210000\ MPa$

$t_f = thickness\ of\ face\ plate = 2.8\ mm$

$t_c = thickness\ of\ core = 50\ mm$

$\upsilon = possion's\ ratio = 0.3$

$n = number\ of half sine\ in\ transverse\ direction$

$m = number\ of half sine\ in\ logitudinal\ direction$

Flexural stiffness for an isotropic sandwich panel,

$$D_1 = D_2 = D_3 = \frac{E_f t_c^2 t_f}{2(1-\upsilon^2)} = 8.08 * 10^8$$

Critical buckling load per unit width can be,

$$N_{cr} = -\frac{\pi^2 a^2}{m^2}\left[D_1\left(\frac{m}{a}\right)^4 + 2D_3\left(\frac{m}{a}\right)^2\left(\frac{n}{b}\right)^2 + D_2\left(\frac{n}{b}\right)^4\right] = 1808.35\ N/mm$$

Critical stress,

$$\sigma_{cr,sandwich,analy} = \frac{N_{cr}}{2t_f} = 332.92\ MPa$$

$$\lambda_p = \sqrt{\frac{f_y}{\sigma_{cr,sandwich,analy}}} = 1.85$$

As per winter formula,

$$\rho = \frac{1}{\lambda_p}\left(1 - \frac{0.22}{\lambda_p}\right) = 0.48$$

Buckling Resistance,

$$N_{bRd} = 2t_f b f_y \rho = 12699.7\ KN$$

D.4 FEM

Dimensions and geometry of the sandwich panel are taken from chapter 4. Plan i.e. length and width of the sandwich panel is same as that of the stiffened plate. The only varying parameter is thickness, which comprises of thickness of the faceplates and thickness of the core.

Mesh & Element type:-

For modelling in ANSYS, SOLID186 element type is used. SOLID186 is a second-order 3-D 20-node solid element that exhibits quadratic displacement behaviour. It has 20 nodes having three degrees of freedom per node (x, y and z-direction). SOLID186 is an element which offers the ability to model local bending effects. Because of its quadratic element property, it prevents hour-glassing. It also prevents shear locking.

For materials under pure bending, the shear locking effect is observed. Elements which are exposed to pure bending ideally experience a curved shape change. Linear elements are unable to experience this change. In the case of linear materials, incorrect artificial shear stress is introduced. Because of this shear deformation is generated instead of bending deformation. Overall effect is that under bending moment linear fully integrated components become overly stiff or locked. This shear locking is not observed in the quadratic element due to the introduction of additional nodes.

The computational efficiency is decreased which is an undesirable effect due to the presence of additional nodes. This can be minimized by using a reduced integration solution. Reduced integration may result in an excessively flexible element. This is also known as hourglassing effect. Because of hourglassing, meaningless results are produced. This is due to the fact that normal and shear stresses at point of integration are assumed to be zero. In the through-thickness direction of the panel, if a single layer of elements is used then hourglassing may occur in solid elements of second order. In this study, multiple elements are used in the through-thickness direction so as to prevent hourglassing.

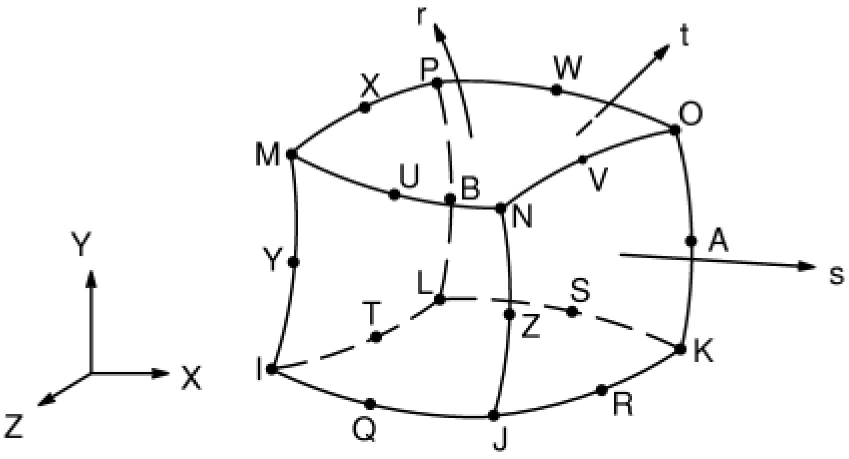


Figure D.10 ANSYS element type SOLID186 [9]

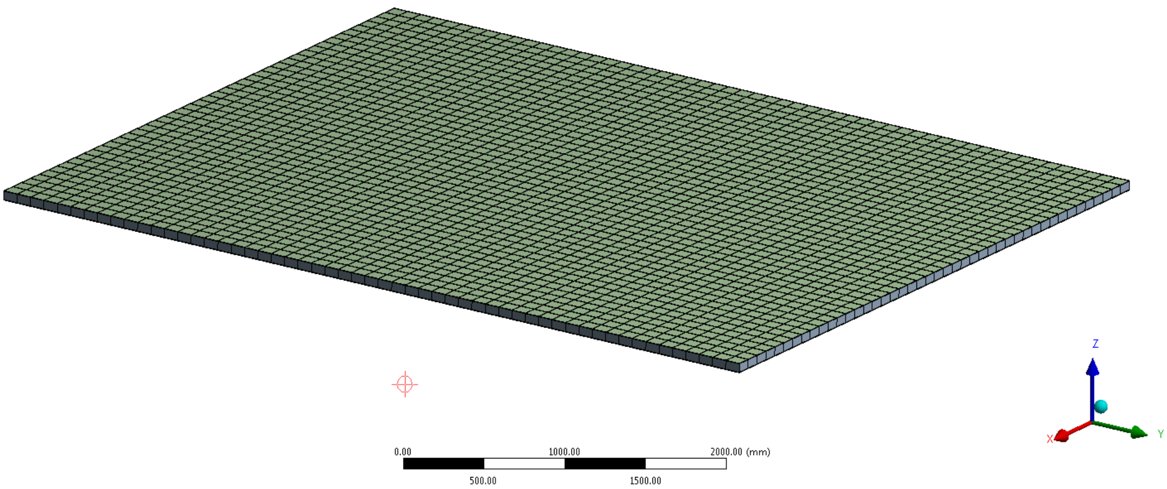


Figure D.11 Sandwich Meshing.

Boundary & Loading Conditions:-

The boundary condition of the sandwich panel is illustrated in flowing figure. The bottom edge of the sandwich panel is constrained from all degrees of freedom except rotation around x-axis. Whereas at top edge rotation around x-axis and displacement along the y-axis is, free and all other freedoms are constrained. Displacement of side edges in the direction perpendicular to the plane is restricted i.e. displacement in the z-direction is restrained.

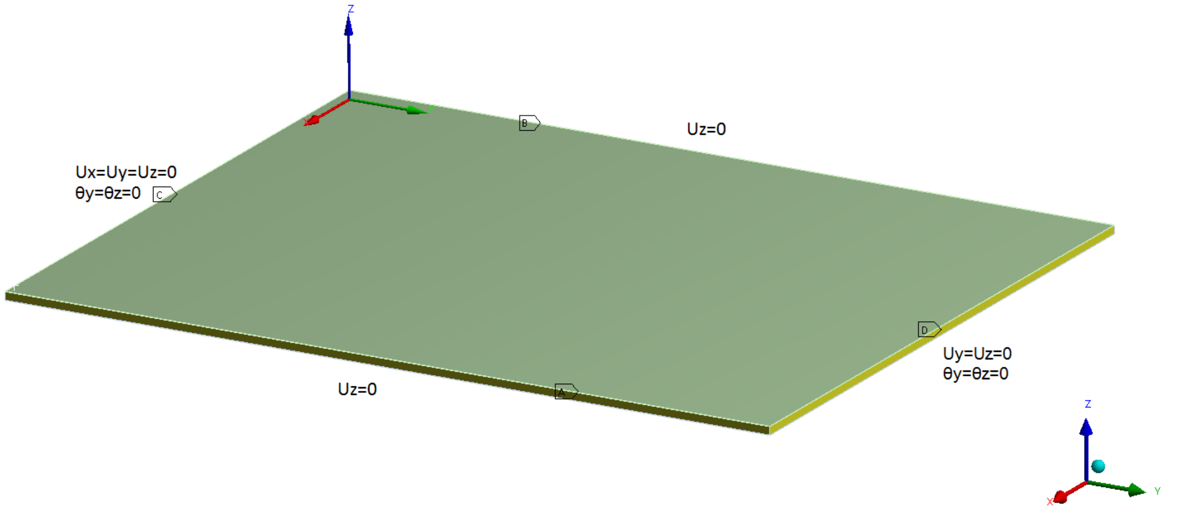


Figure D.12 Boundary conditions of sandwich panel

The sandwich plane is loaded with in-plane compression along the y-axis. The load is applied only on faceplate since the contribution of core to buckling is assumed negligible. Load of 1000 Newton is applied. The following figure shows loading on the sandwich panel.

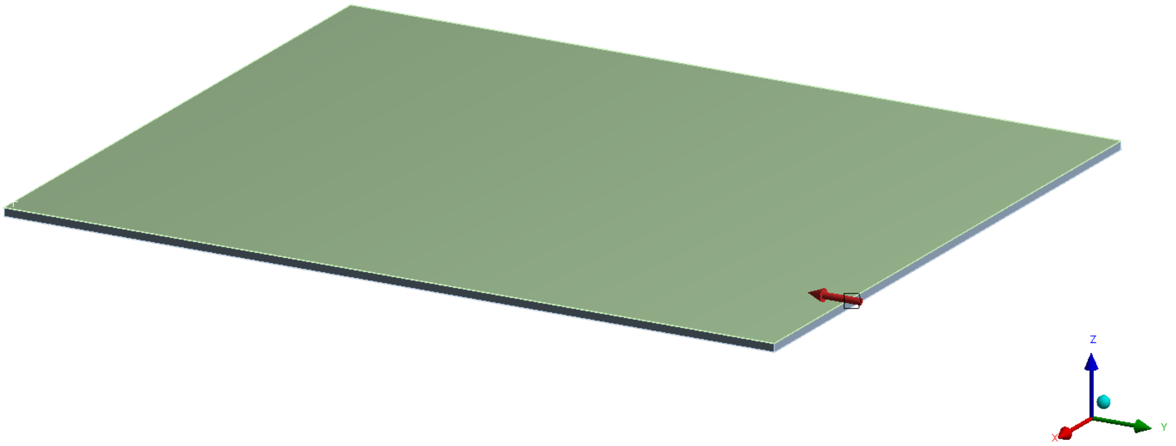


Figure D.13 Loading Condition of sandwich panel

Result of FEM:-

Eigenvalue buckling analysis is done on the sandwich panel to find load multiplication factor. Load multiplication factor will give a value of the load at which sandwich will buckle. As per the theory, the first buckling mode is critical in the case of flexural buckling analysis. Generally, in Eigen buckling analysis, it is customary to calculate at least 10 buckling mode but load multiplier value corresponding to the first mode is only important from point of view of flexural buckling. Also, it is important to find out the value of stress at the buckling load. For this minimum principal, stress is calculated. The following figure shows first Eigen buckling mode and load multiplier corresponding to it.

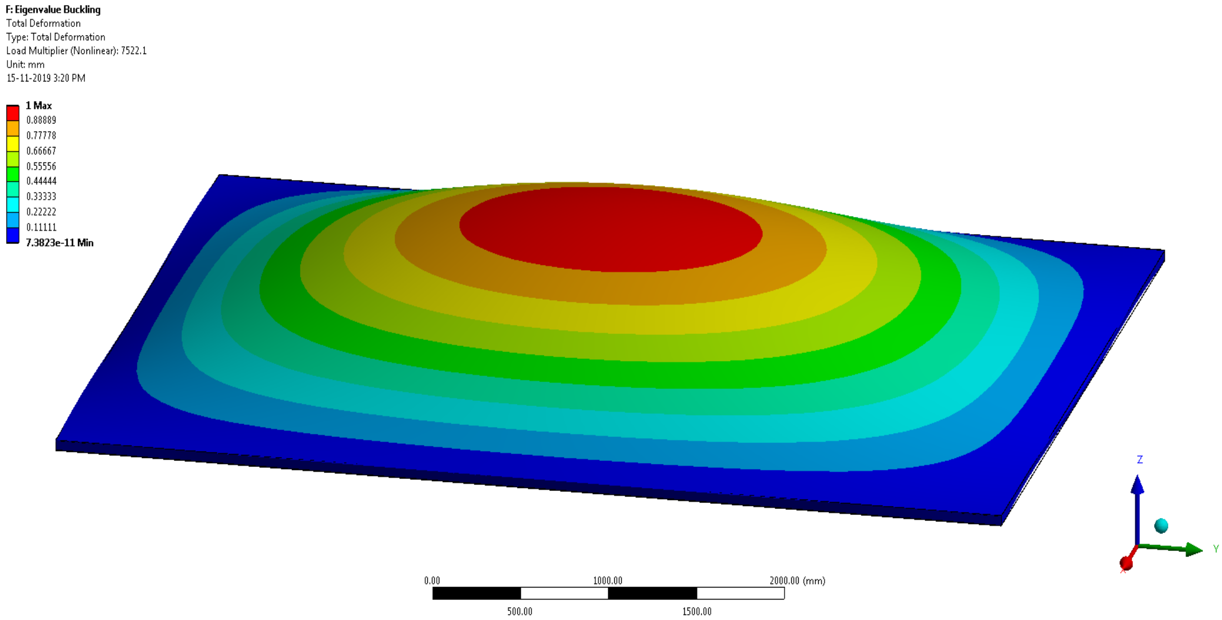


Figure D.14 First Eigen buckling load calculated with ANSYS

$$Load\ Multiplier = 7522.1$$

$$Buckling\ load = load\ multiplier * applied\ load$$

$$Buckling\ load = 7522.1 * 1000 = 7522.1\ KN$$

Now, from the above figure position of maximum displacement can be observed. After this, minimum principal stress at the same position will be measured but since it is difficult to exactly point out this position, some points are arbitrarily chosen near the point of maximum deformation and, minimum principal stress at all these points will be measured. The following figure will give an idea of this.

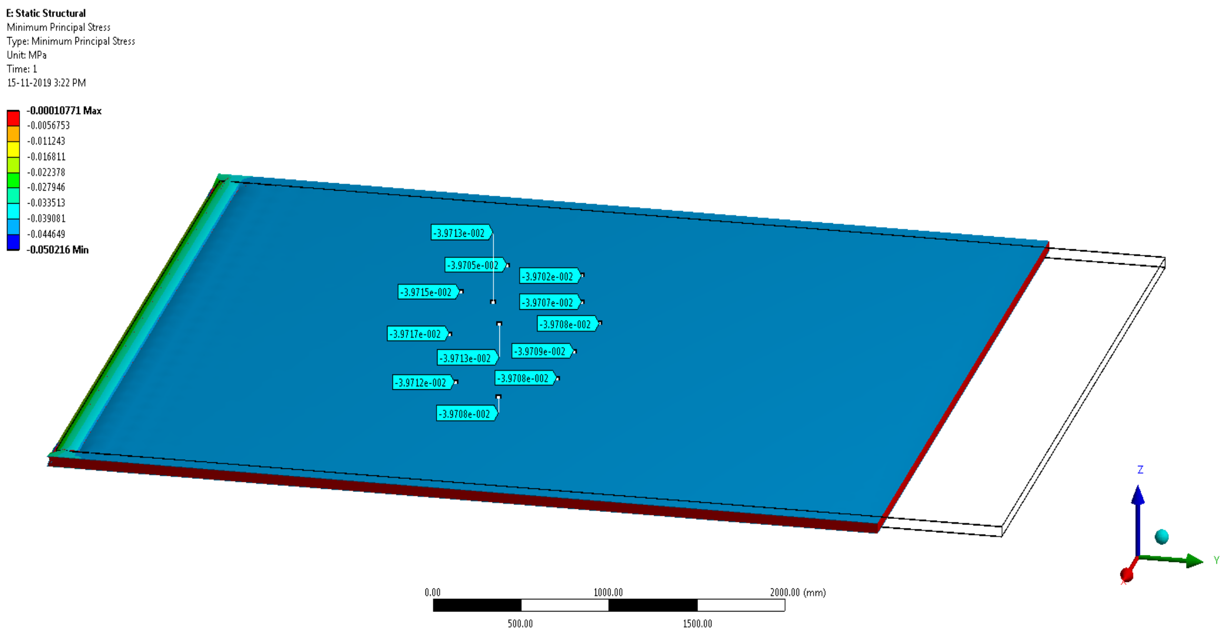


Figure D.15 Minimum principle stress calculated with ANSYS

It can be observed that faces are in uniform compression so the values of minimum principal stress at all points are equal. If values at different points would have been different then it is better to take an average. But in this case, the average will be exactly same as the value at an individual point.

$$minimum\ principle\ stress = 3.97 * 10^{-2}\ MPa$$

Form these two value i.e. load multiplier and minimum principal stress, stress at buckling can be calculated as follows,

$$stress = load\ multiplier * minimum\ principle\ stress$$
$$stress = 7522.1 * 3.97 * 10^{-2}$$
$$stress = 298.62\ MPa$$

Another simple method that can be applied in this case, to calculate stress at buckling, would be dividing buckling load with cross-section area on which load is applied.

$$\sigma_{cr,Sandwich,FEM} = \frac{F_{FEM}}{A} = \frac{F_{FEM}}{2bt_f} = \frac{7522.1 * 10^3}{2 * 2.8 * 4320} = 310.9\ MPa$$

To find the resistance of sandwich panel non-linear analysis of sandwich panel is performed. In this non-linear analysis rather than applying a load, displacement is applied and the resultant force is calculated. The displacement is applied in short increment until the point where it is not possible to achieve force convergence and model fails. The displacement increment & force reaction corresponding to it is arranged in tabular form. The maximum force reaction can be treated as a load-carrying capacity of the sandwich panel. In addition, to visualise this, graph between displacement increment and corresponding force reaction is plotted as follows,

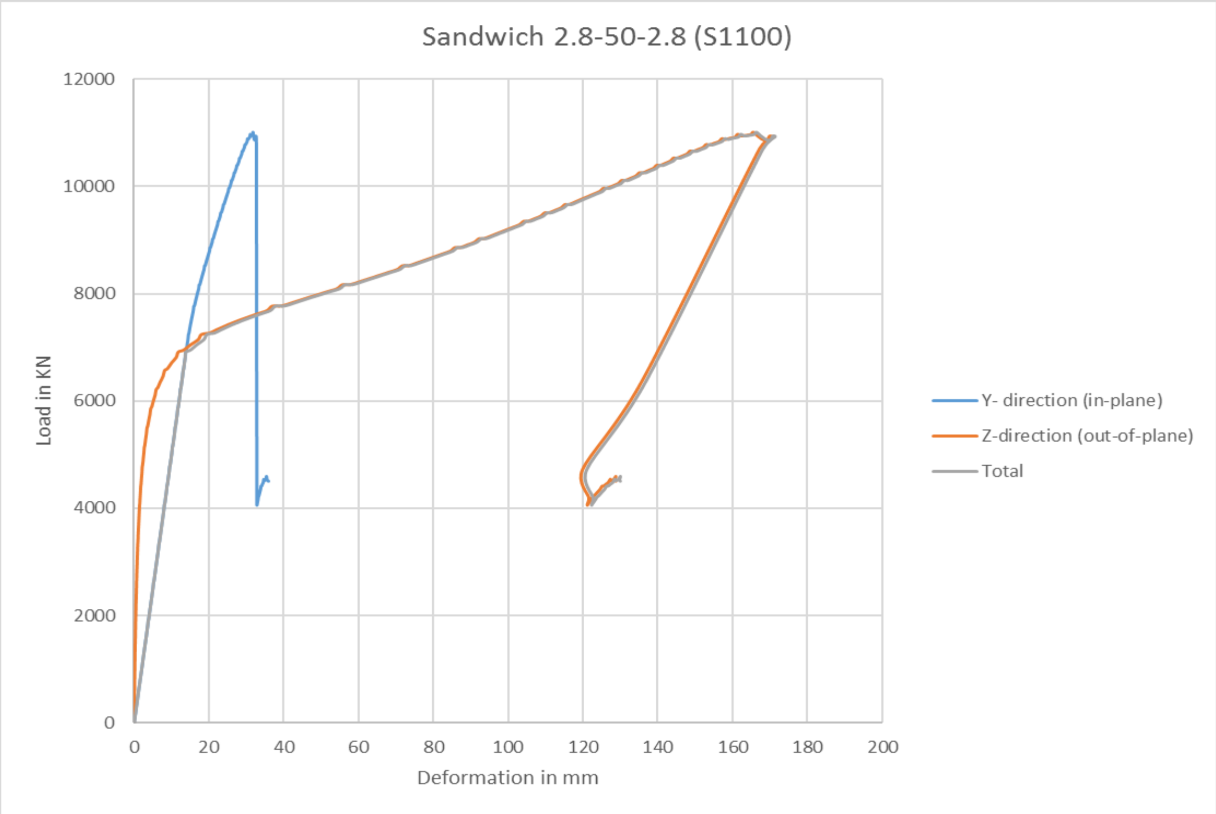


Figure D.16 Nonlinear Buckling Analysis of Sandwich S1100

From the Ansys nonlinear analysis of sandwich S1100, the load-carrying capacity of the sandwich panel comes out to be 11012 KN. This can be observed in the above graph.

Appendix E Calculation of Weight of Sandwich Panel

In the case of a sandwich panel, weight comprises of the weight of faceplates and weight of the core. Faceplates are made from steel whereas core is of aluminium foam. Consider the sandwich panel as shown below for illustration. The sandwich panel has a length of 5500 mm and width of 4320 mm. The thickness of faceplate is 3.6 mm and that of the core is 50 mm. Therefore, the total thickness of the sandwich panel is 57.2 mm.

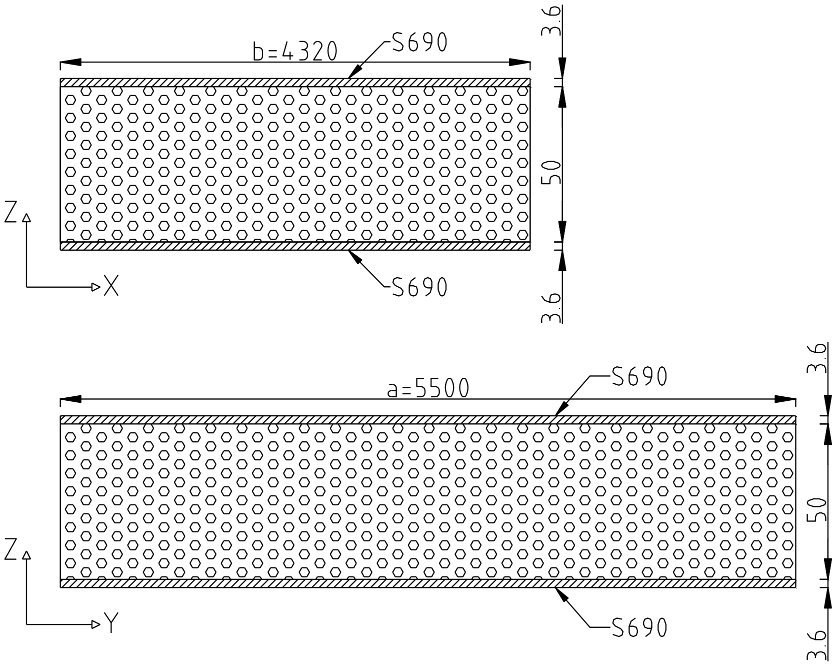


Figure E.1 Sandwich panel 3.6-50-3.6

$a = lenght = 5500\ mm$

$b = widht = 4320\ mm$

$t_f = thickness\ of\ face\ plate = 3.6\ mm$

$t_c = thickness\ of\ core = 50\ mm$

$density\ of\ steel = 7.85 \times\ 10^{-3}\ gram/mm^3$

$density\ of\ aluminum\ foam = 0.7 \times\ 10^{-3}\ gram/mm^3$

Weight of each faceplate,

$$w_{face\ plate} = t_f b \times 7.85 \times 10^{-3} = 122.1\ gram/mm$$

Weight of core,

$$w_{core} = t_c b \times 0.7 \times 10^{-3} = 151.2\ gram/mm$$

Weight of sandwich panel per unit length,

$$w_{sandwich} = 2 \times w_{face\ plate} + w_{core} = 395.4\ gram/mm$$

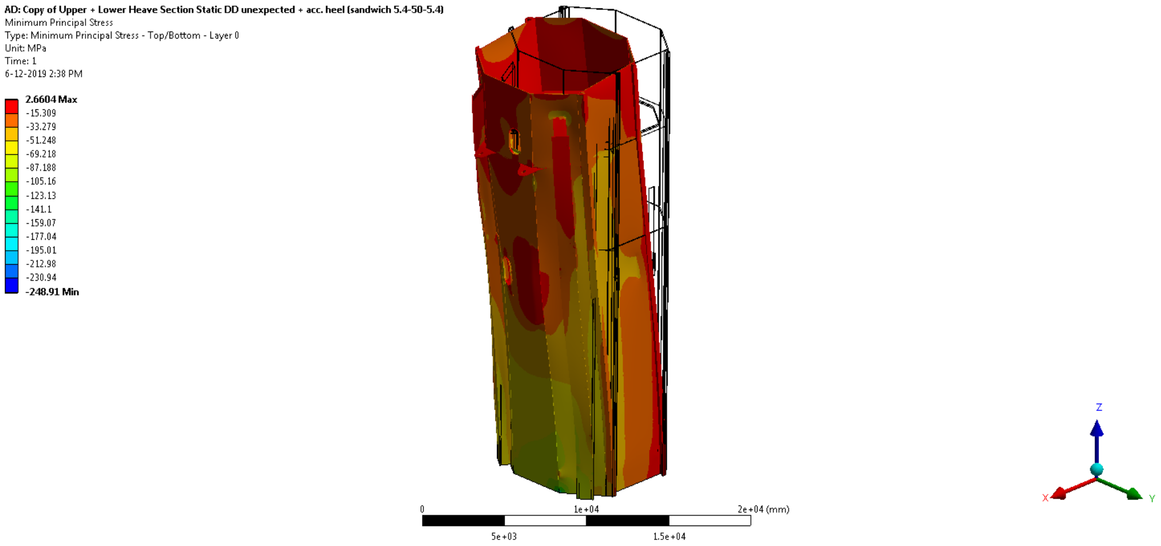
Weight of sandwich panel,

$$W_{sandwich} = w_{sandwich} a = 2175\ KG$$

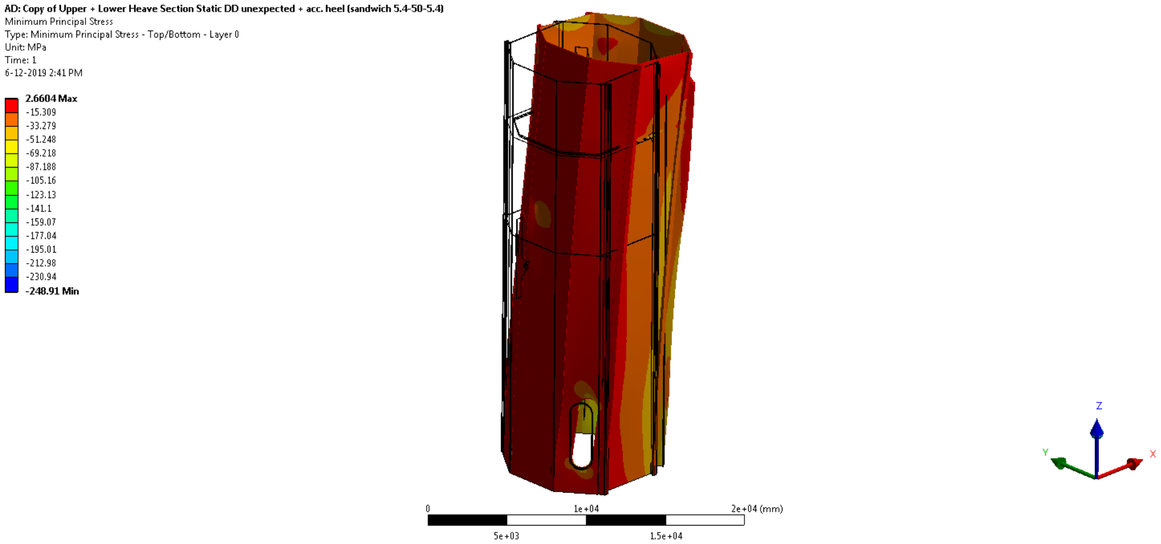
Appendix F Minimum Principle Stress

F.1 Sandwich with Faceplates S355

The minimum principle stress generated in the structure due to different load cases is shown in the following figures.

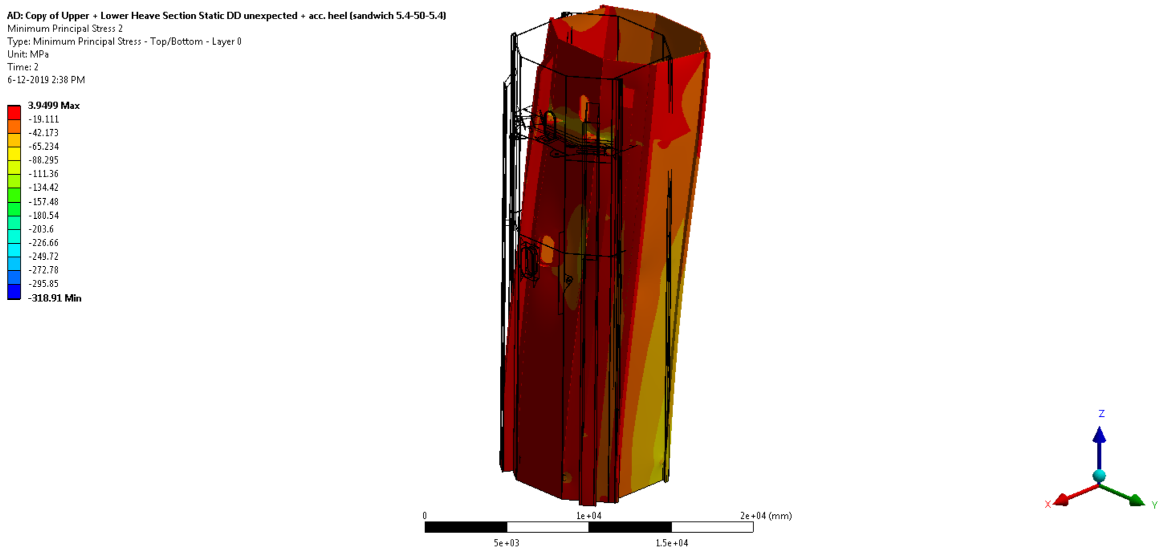


Front View

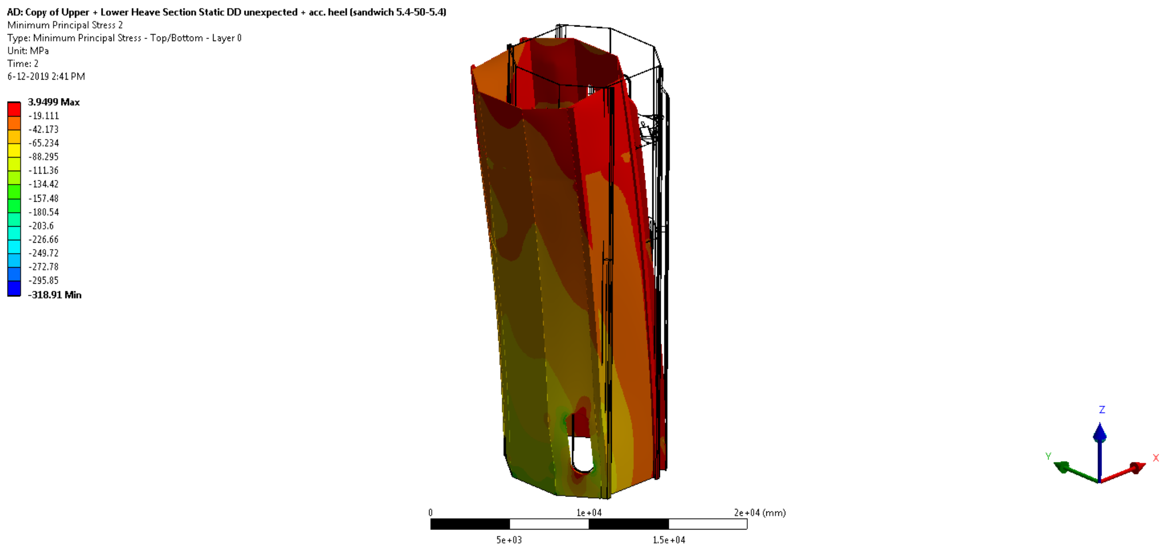


Back View

Figure F.1 Minimum Principle Stress (Load Case 1)

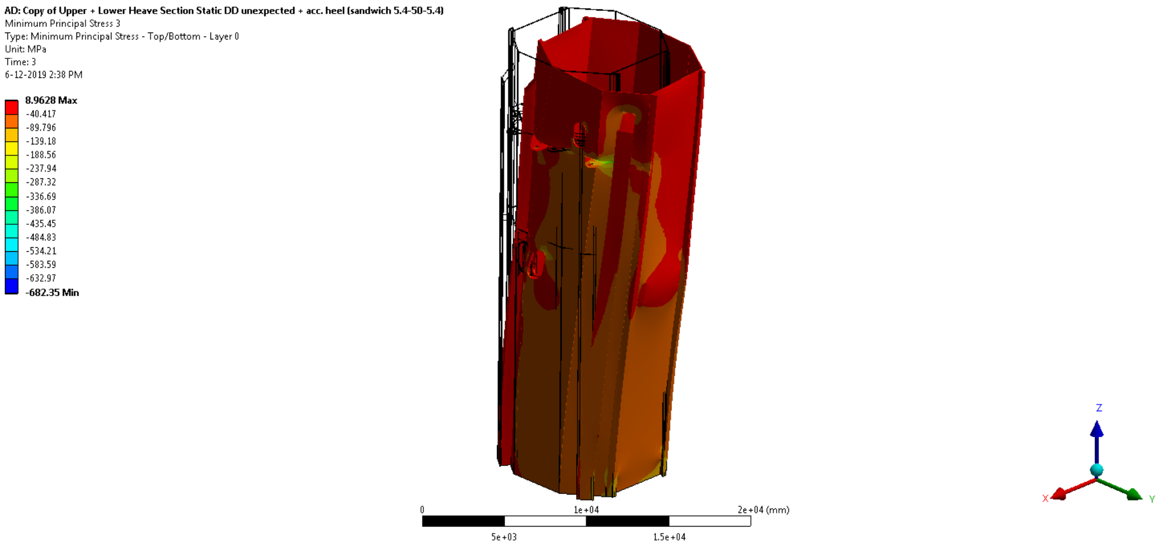


Front View

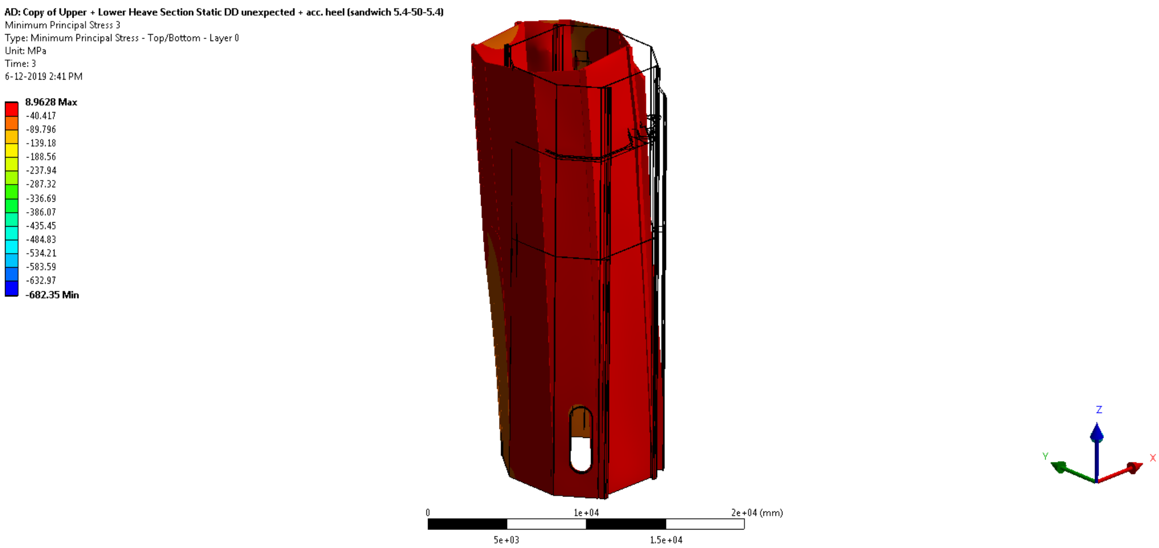


Back View

Figure F.2 Minimum Principle Stress (Load Case 2)

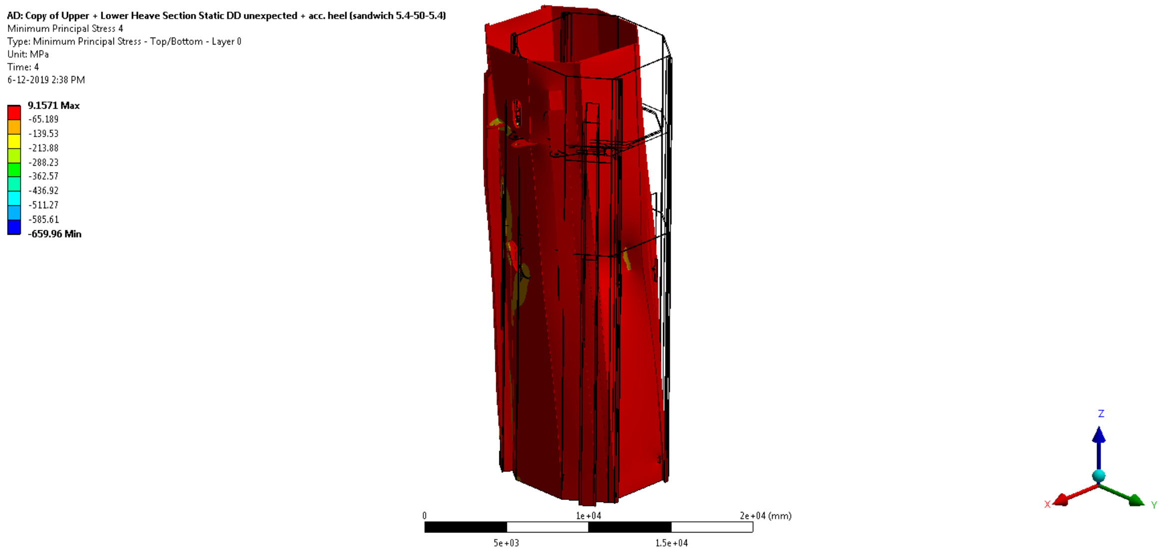


Front View

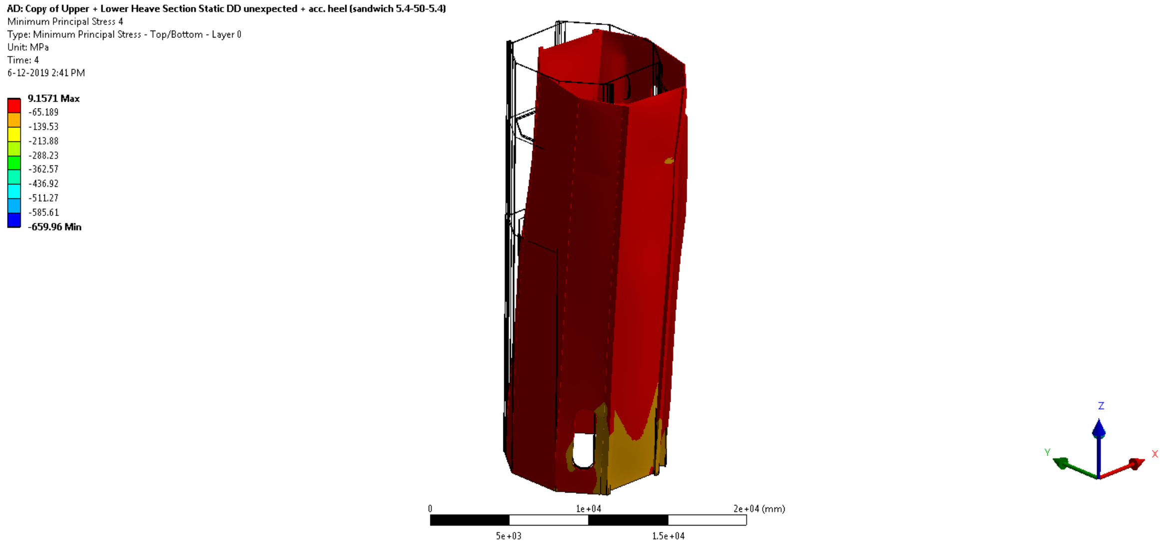


Back View

Figure F.3 Minimum Principle Stress (Load Case 3)

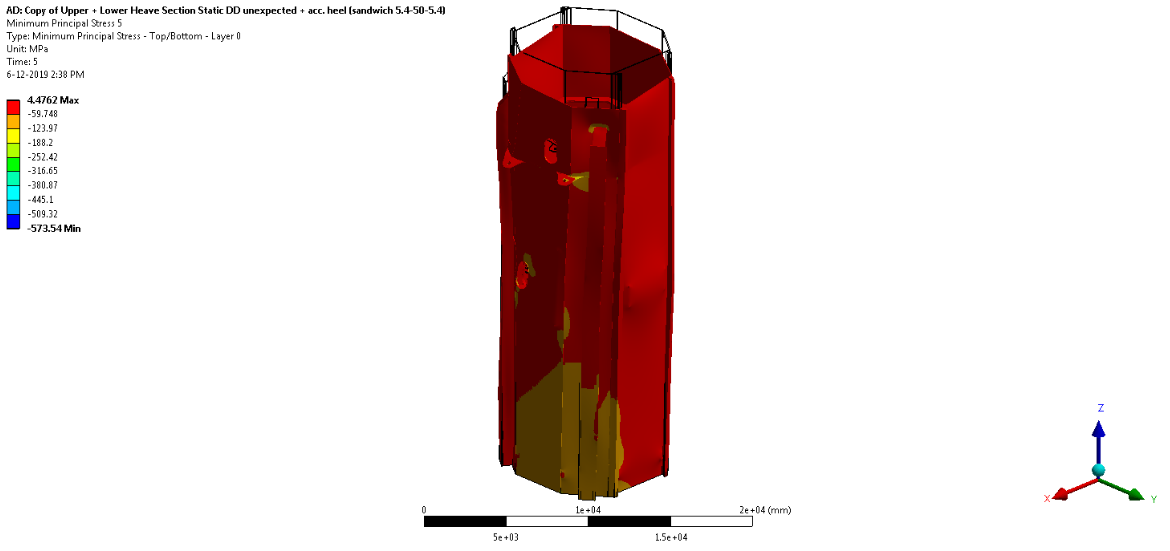


Front View

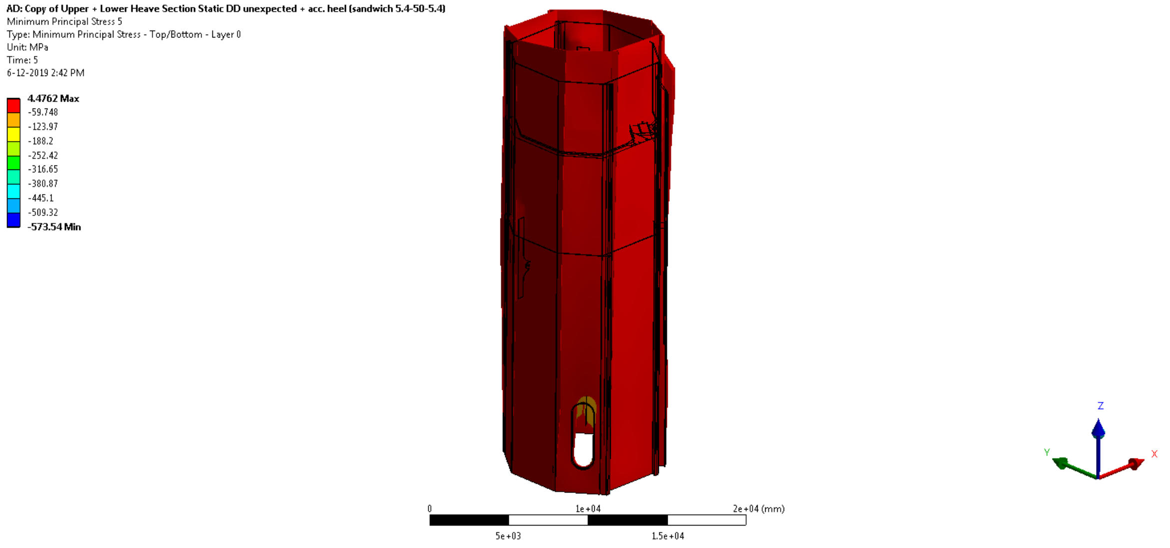


Back View

Figure F.4 Minimum Principle Stress (Load Case 4)

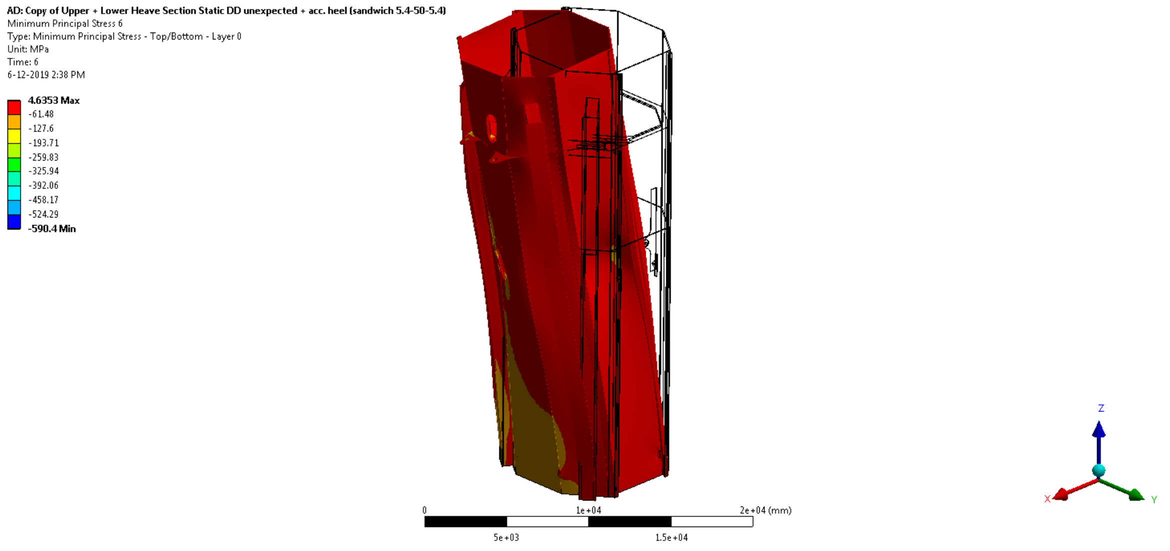


Front View

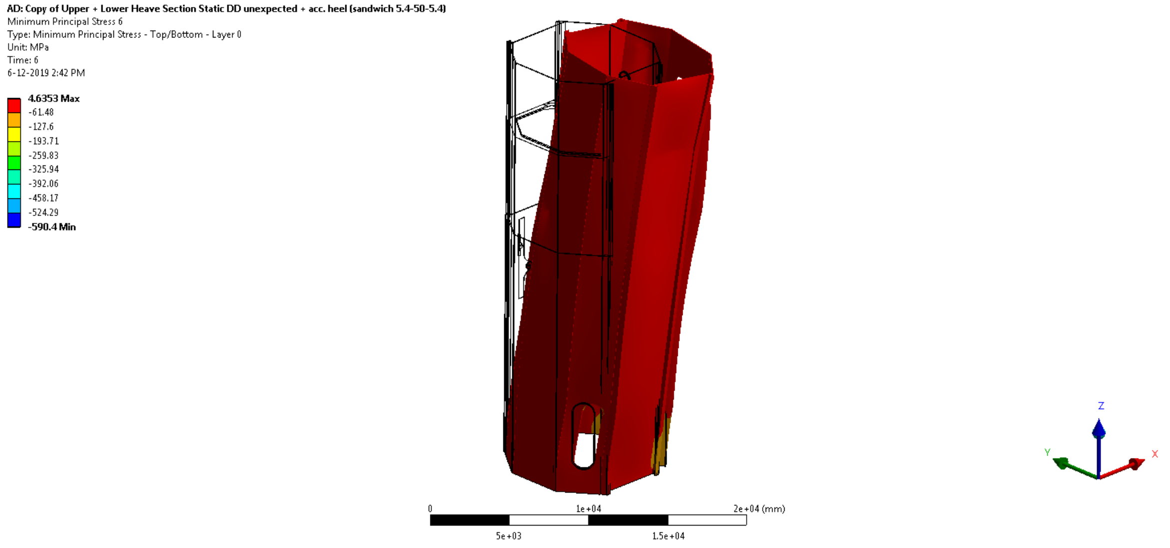


Back View

Figure F.5 Minimum Principle Stress (Load Case 5)

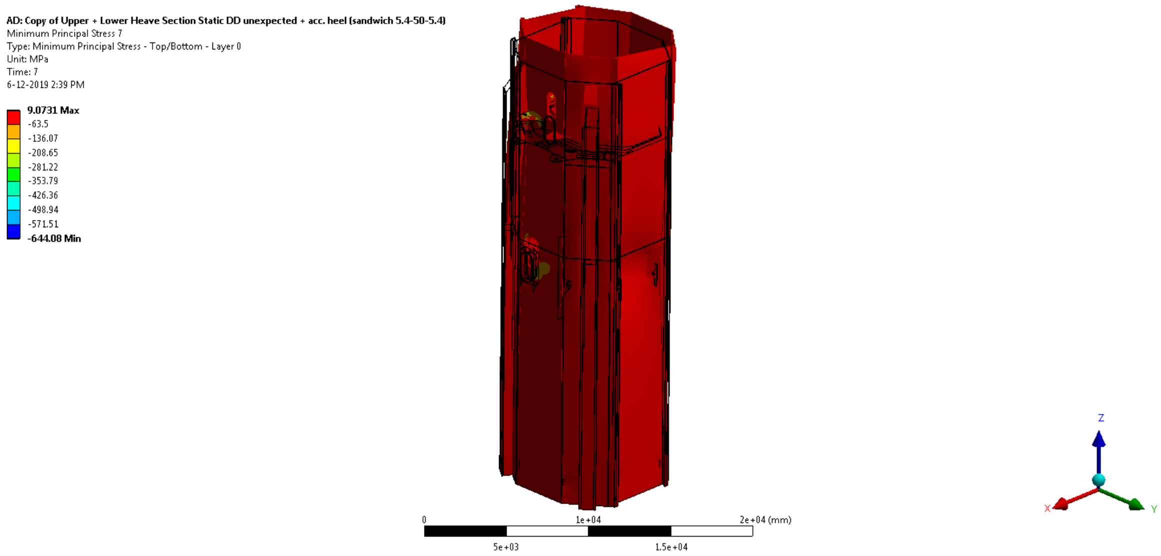


Front View

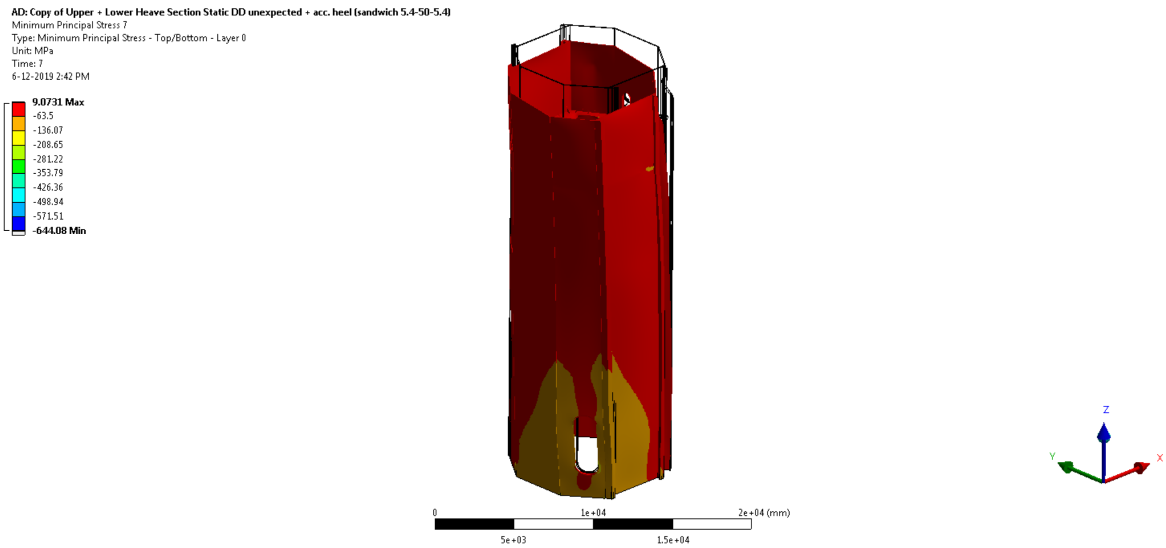


Back View

Figure F.6 Minimum Principle Stress (Load Case 6)

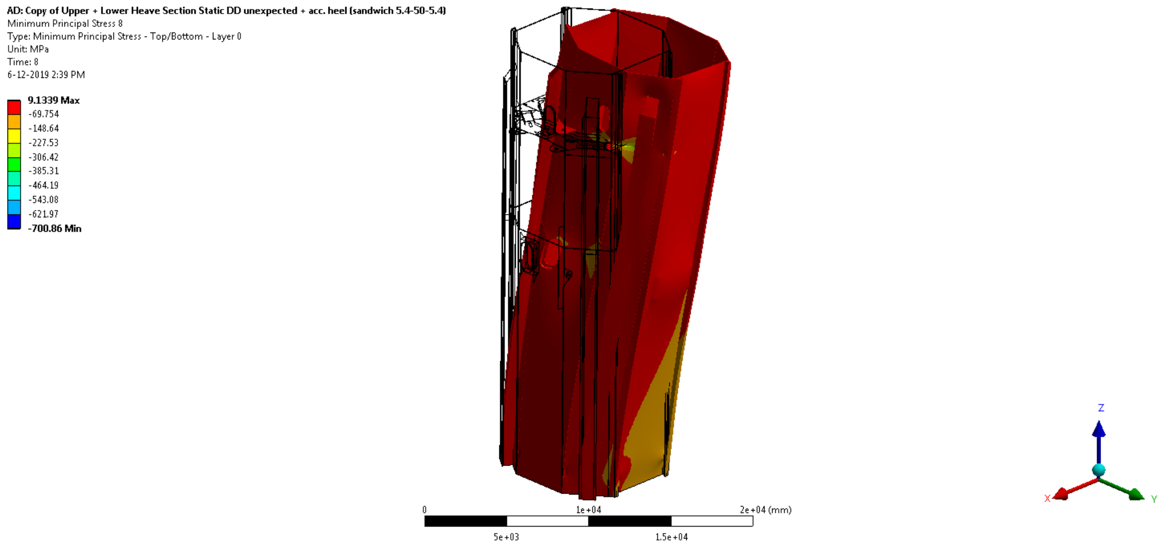


Front View

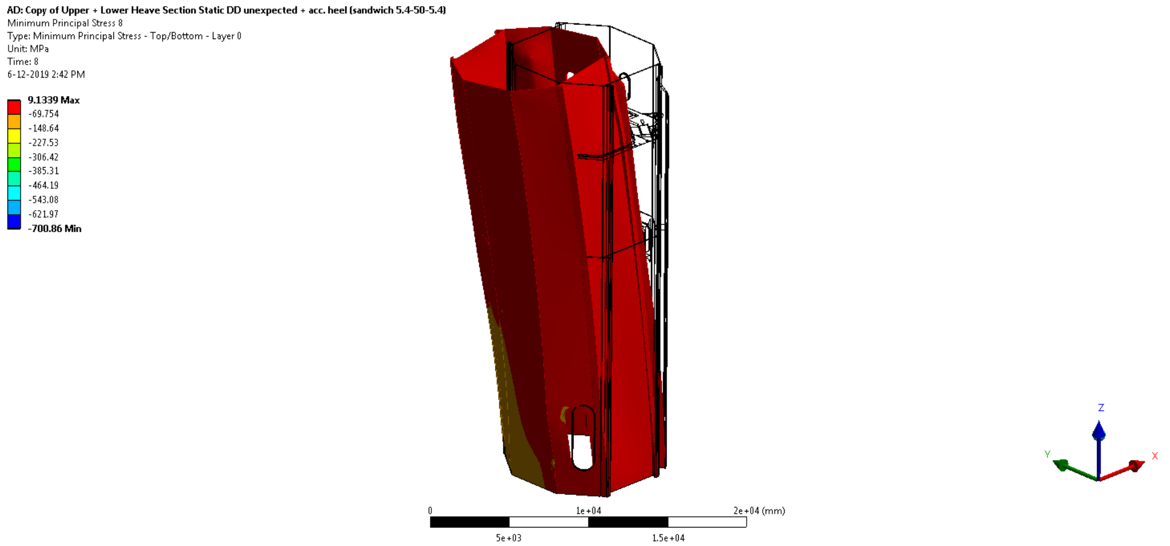


Back View

Figure F.7 Minimum Principle Stress (Load Case 7)



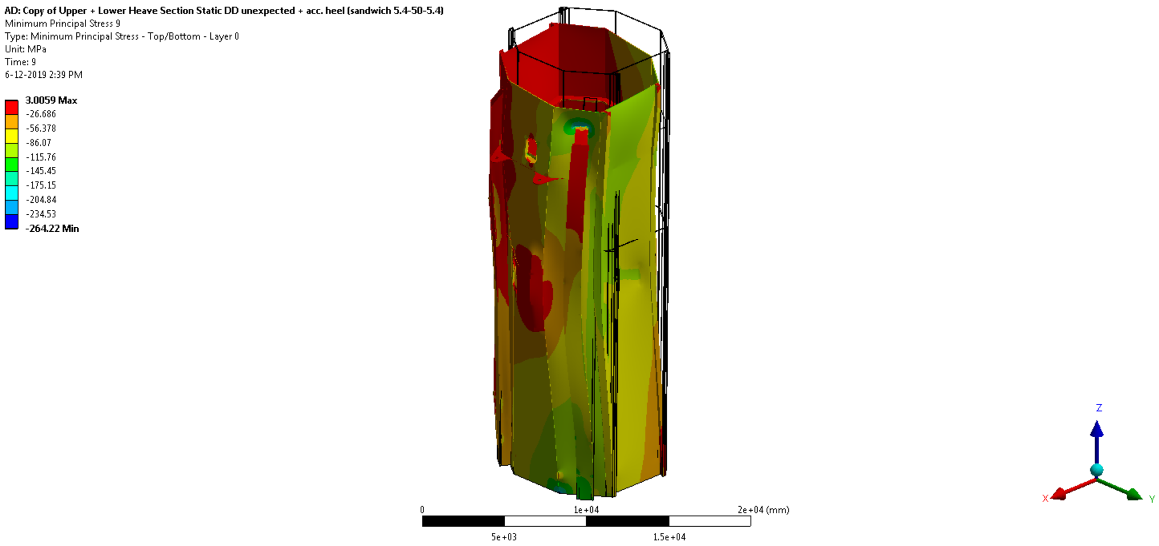
Front View



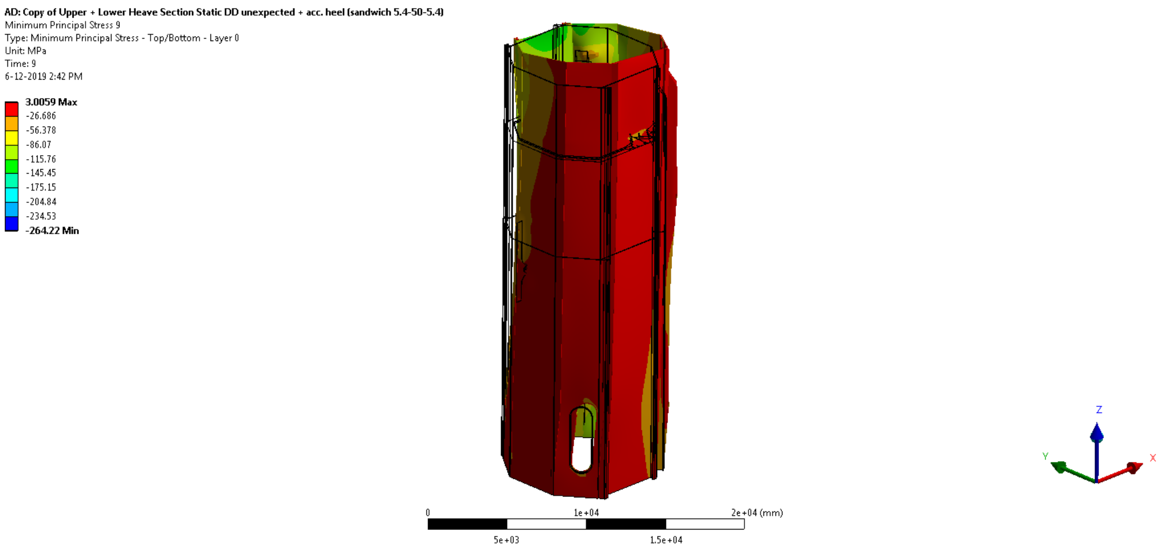
Back View

Figure F.8 Minimum Principle Stress (Load Case 8)

Huisman

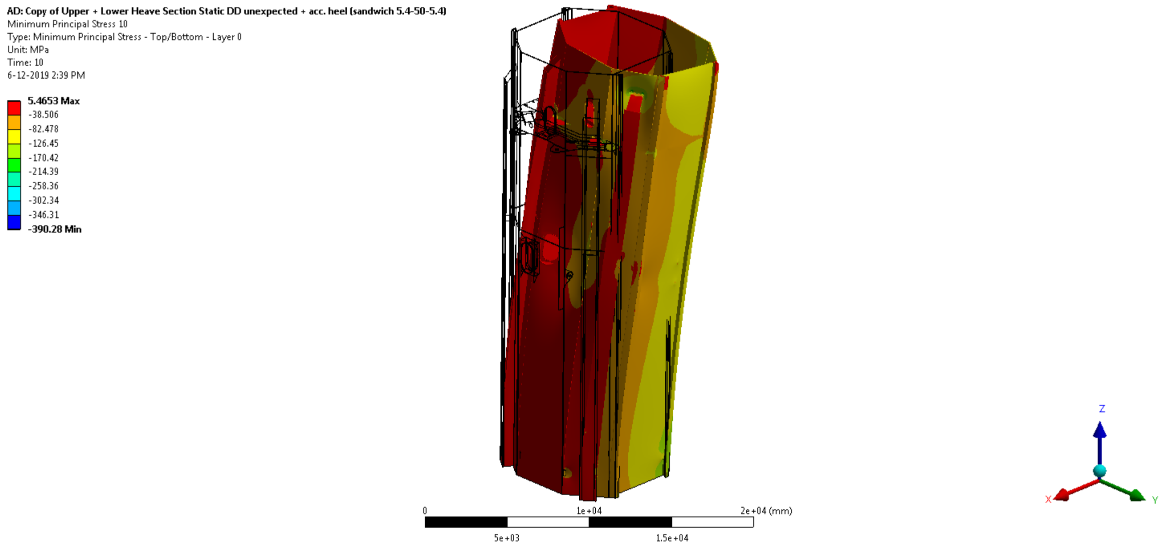


Front View

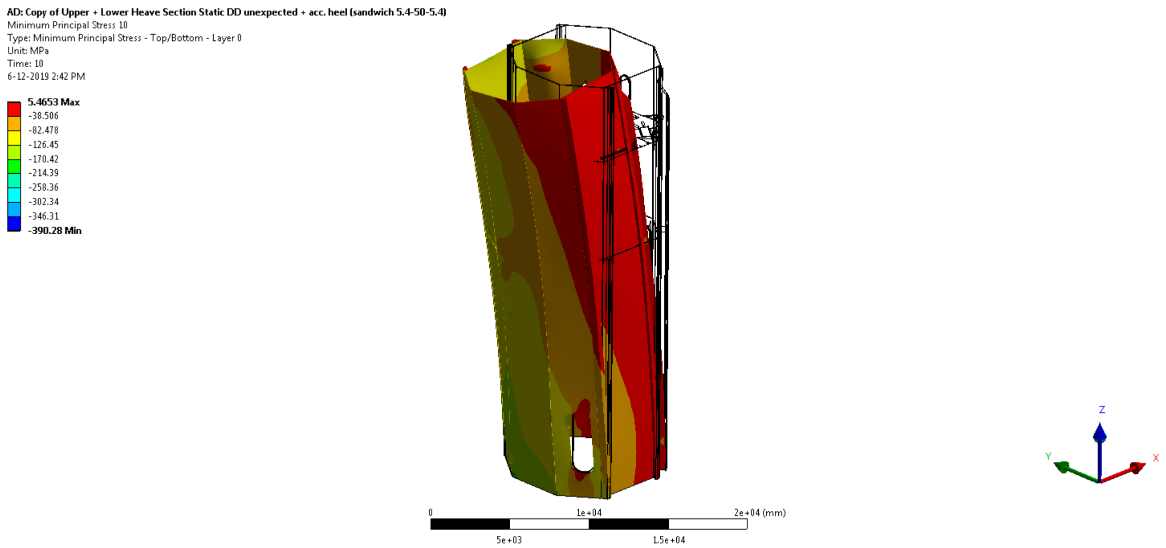


Back View

Figure F.9 Minimum Principle Stress (Load Case 9)

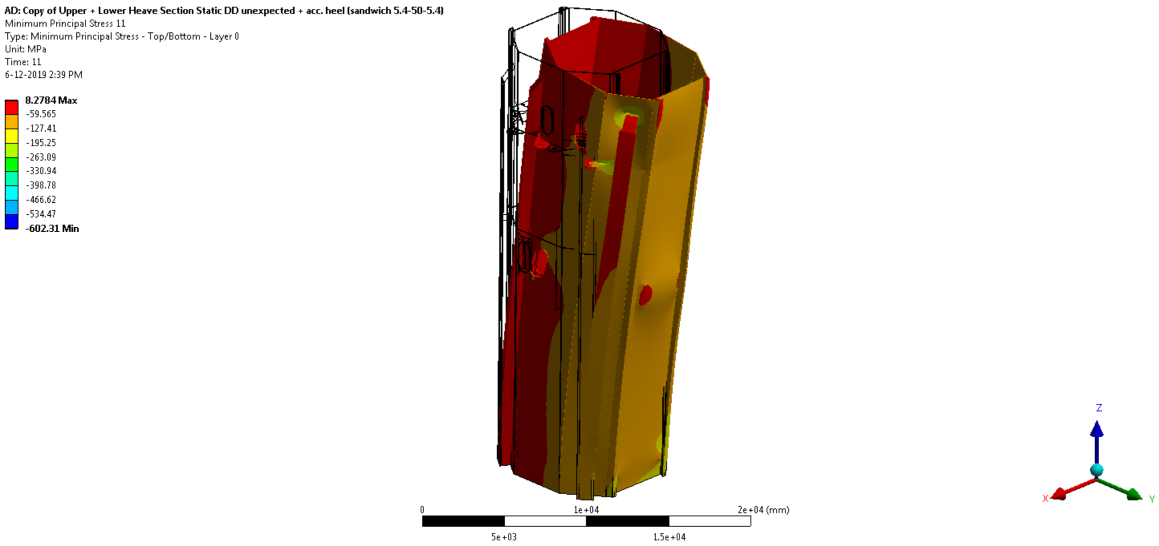


Front View

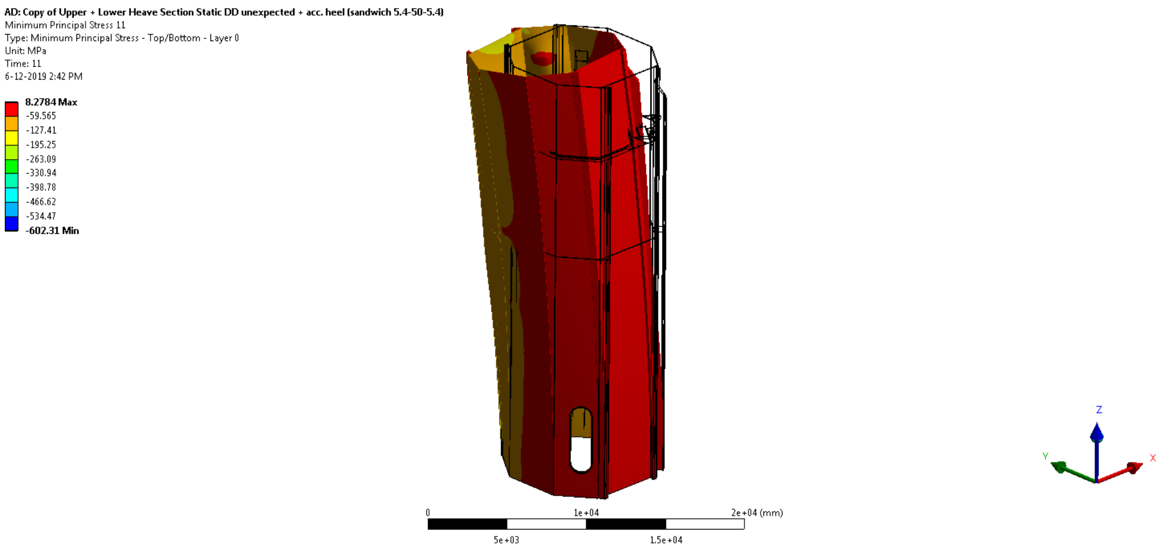


Back View

Figure F.10 Minimum Principle Stress (Load Case 10)

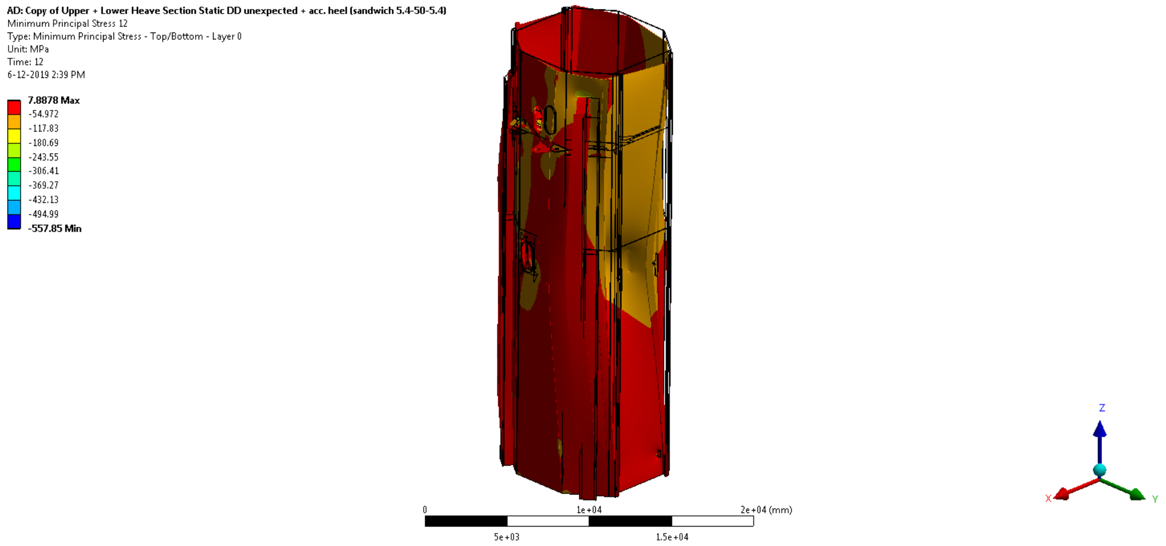


Front View

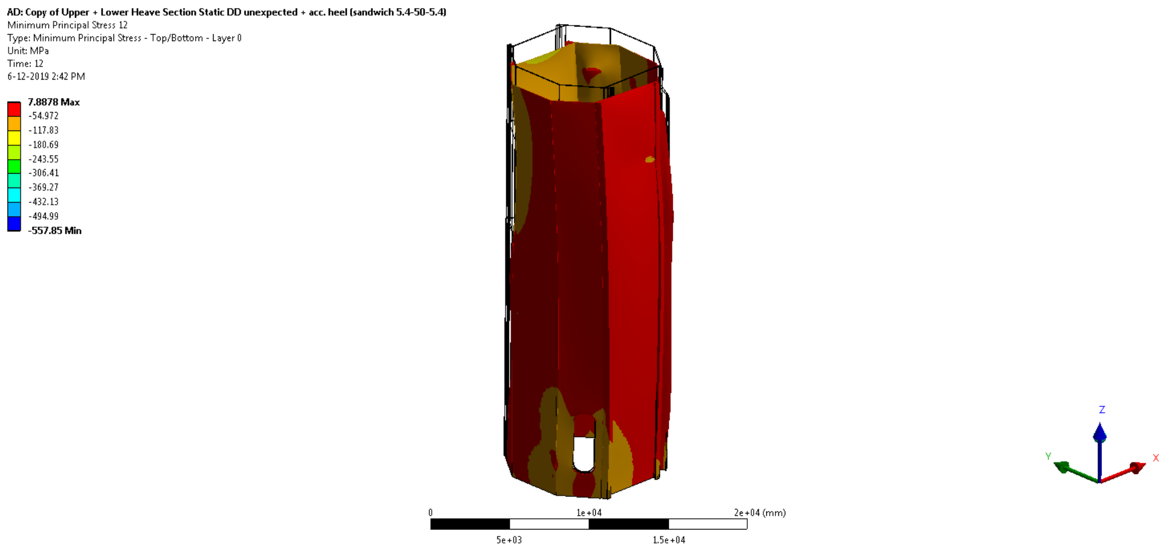


Back View

Figure F.11 Minimum Principle Stress (Load Case 11)



Front View

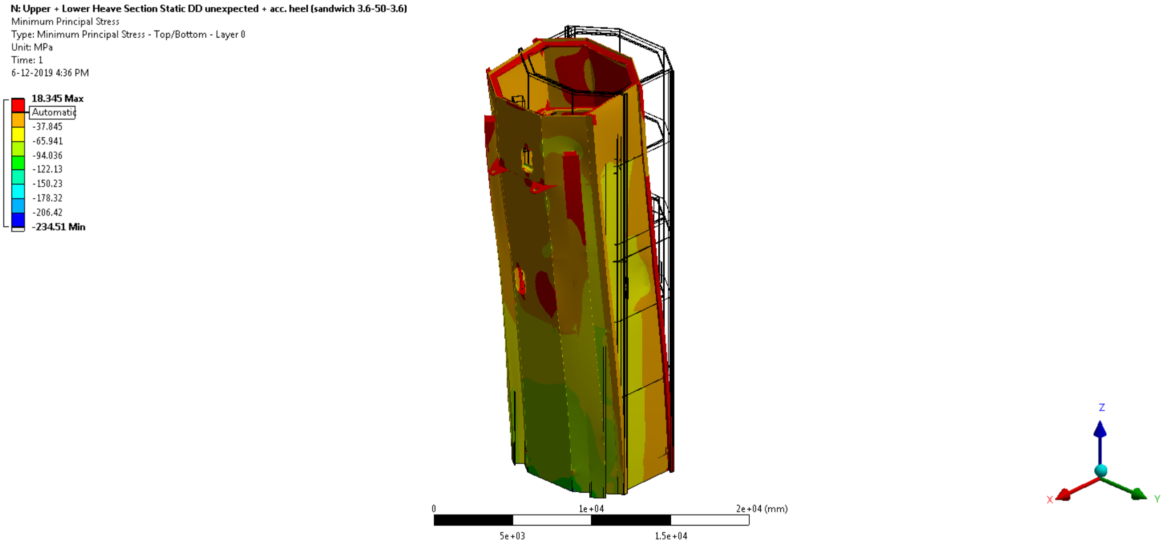


Back View

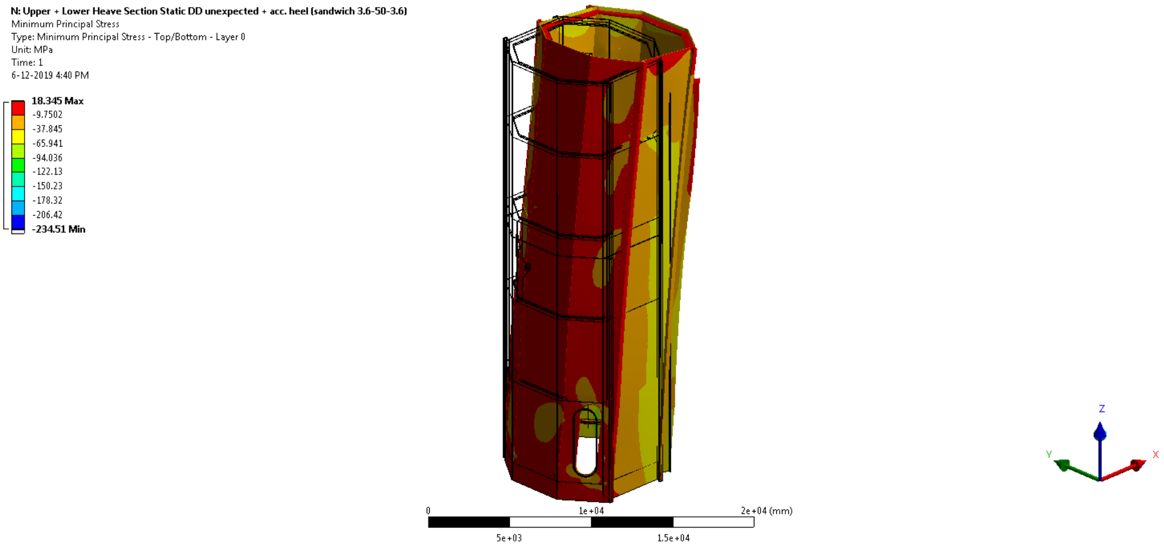
Figure F.12 Minimum Principle Stress (Load Case 12)

F.2 Sandwich with Faceplates S690

The minimum principle stress generated in the structure due to different load cases is shown in the following figures.

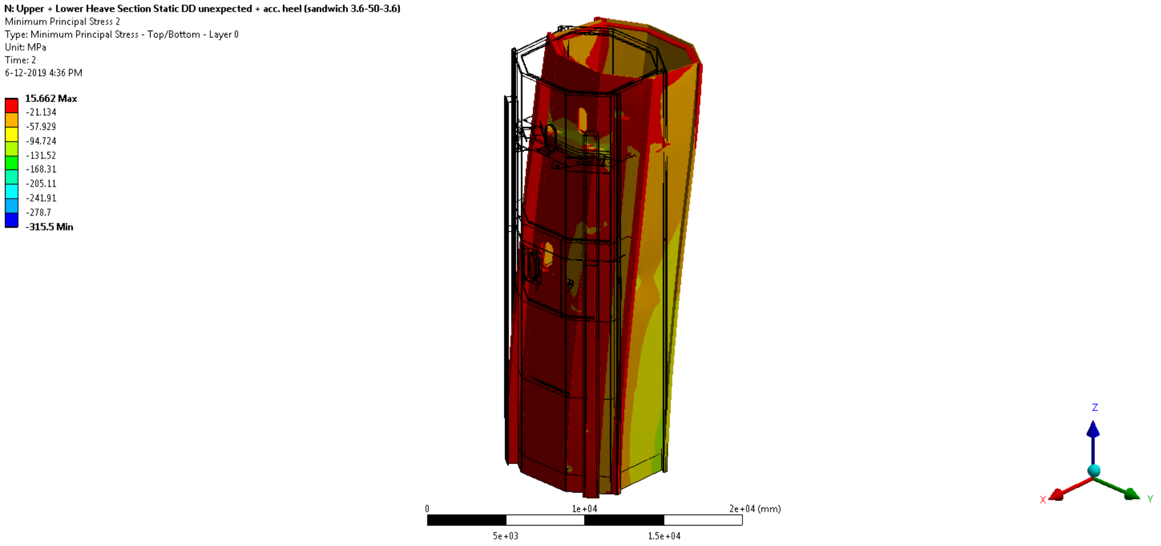


Front View

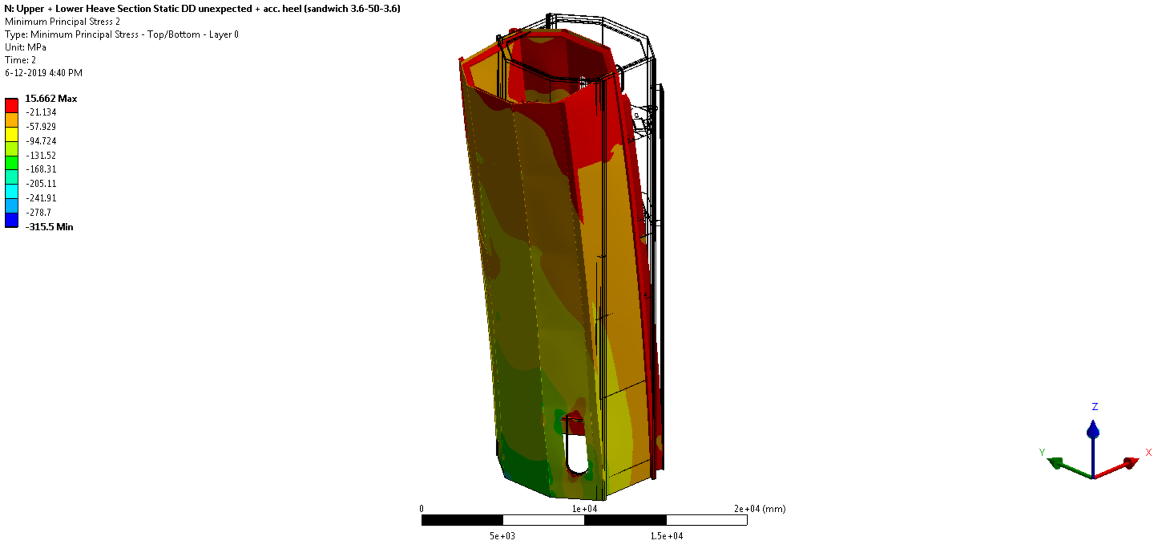


Back View

Figure F.13 Minimum Principle Stress (Load Case 1)

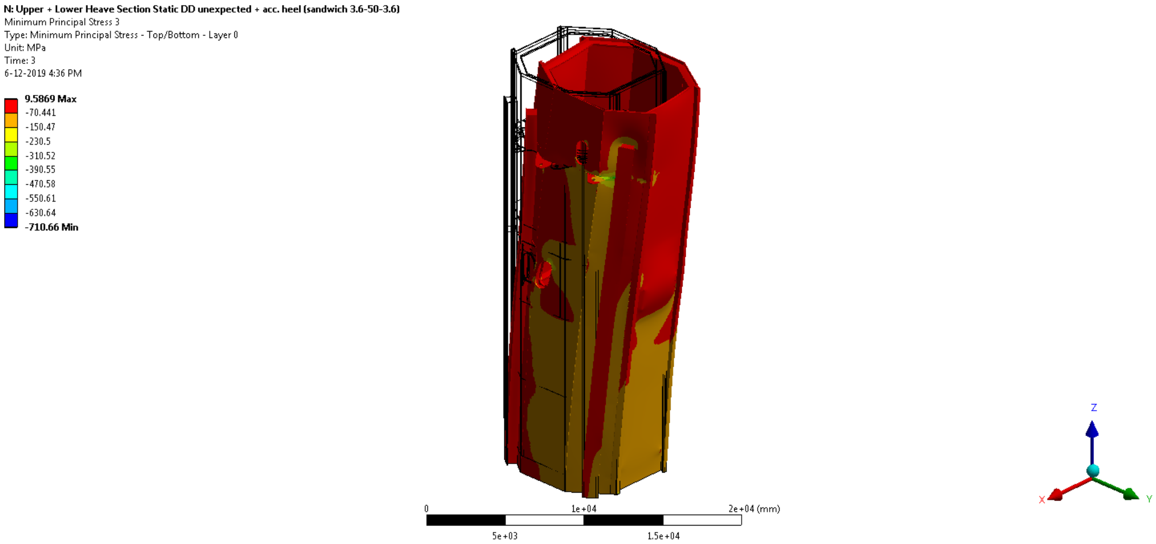


Front View

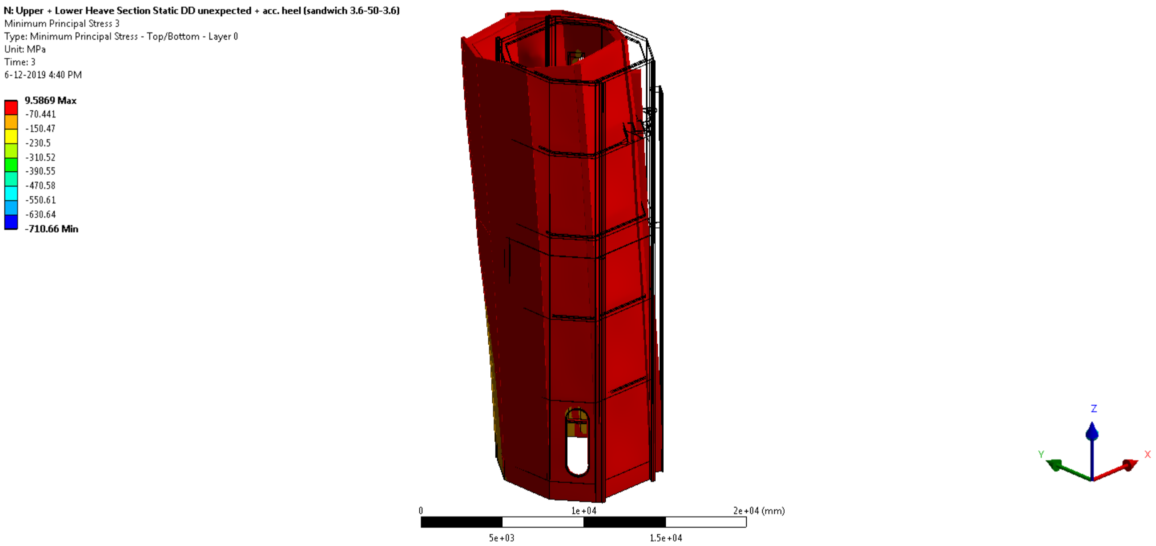


Back View

Figure F.14 Minimum Principle Stress (Load Case 2)

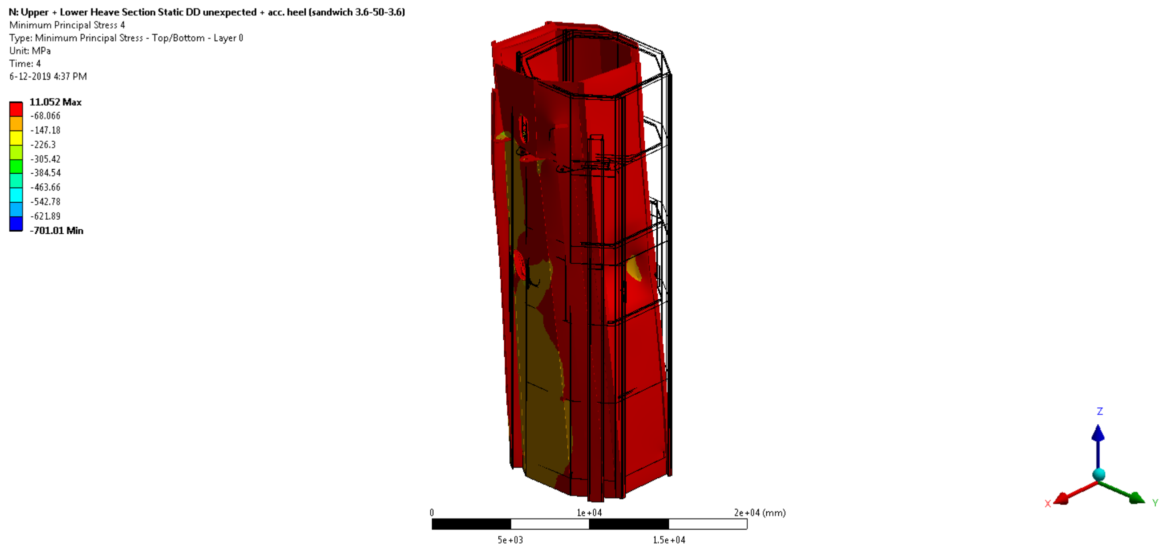


Front View

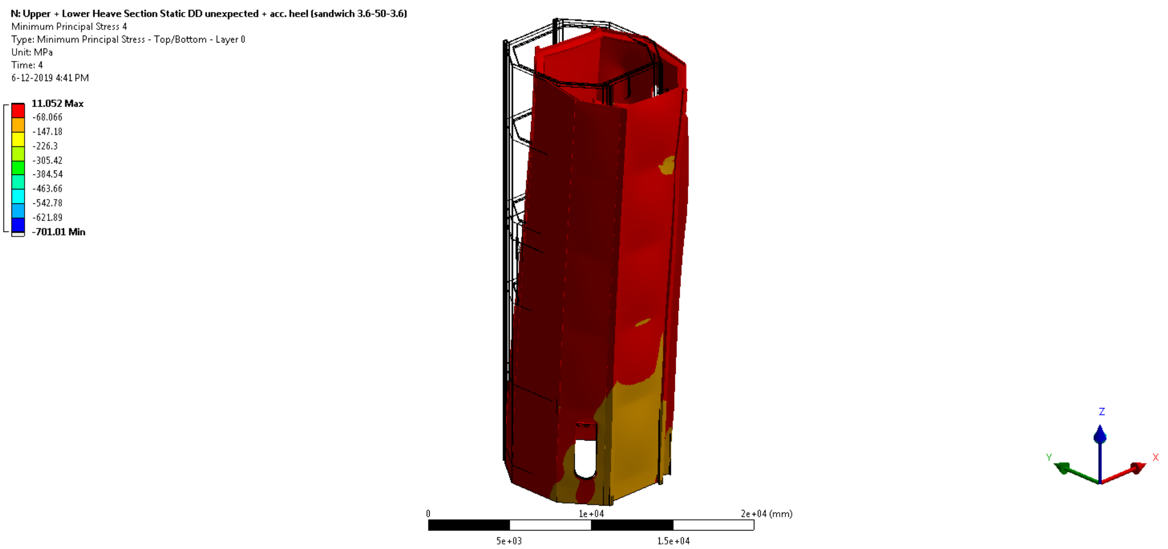


Back View

Figure F.15 Minimum Principle Stress (Load Case 3)

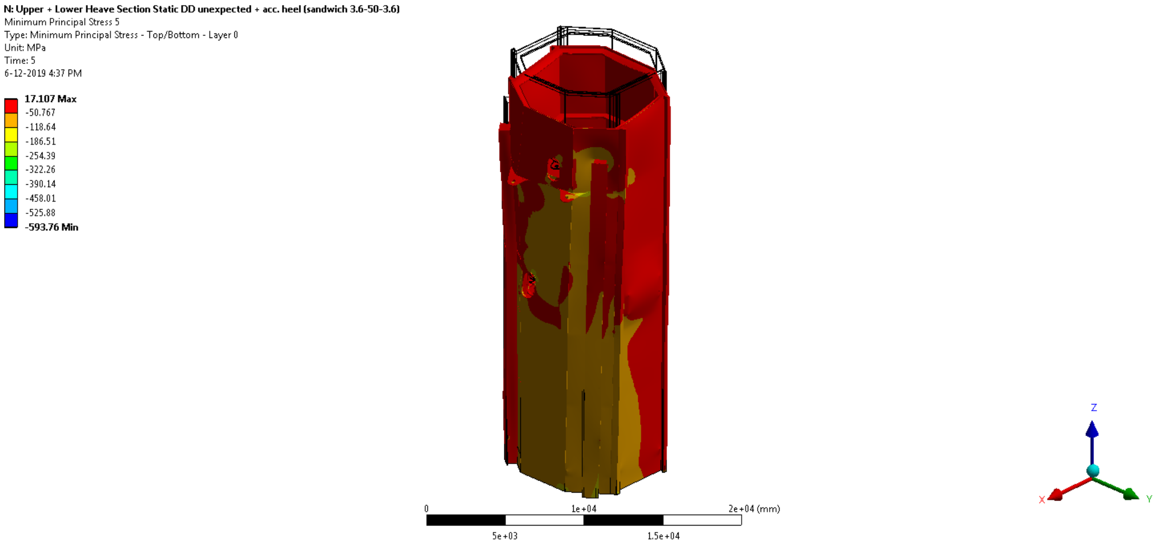


Front View

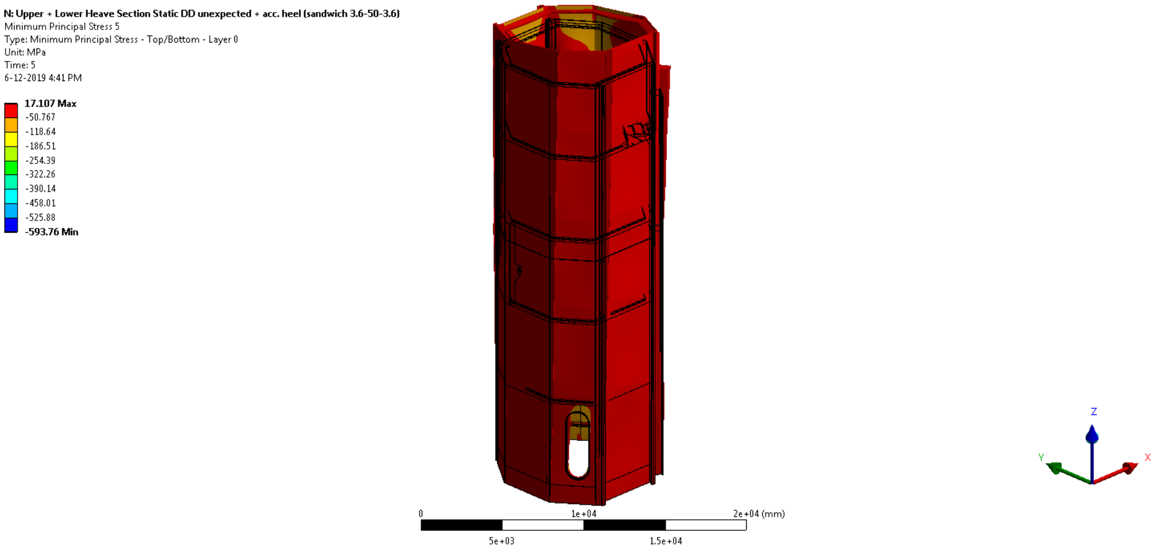


Back View

Figure F.16 Minimum Principle Stress (Load Case 4)

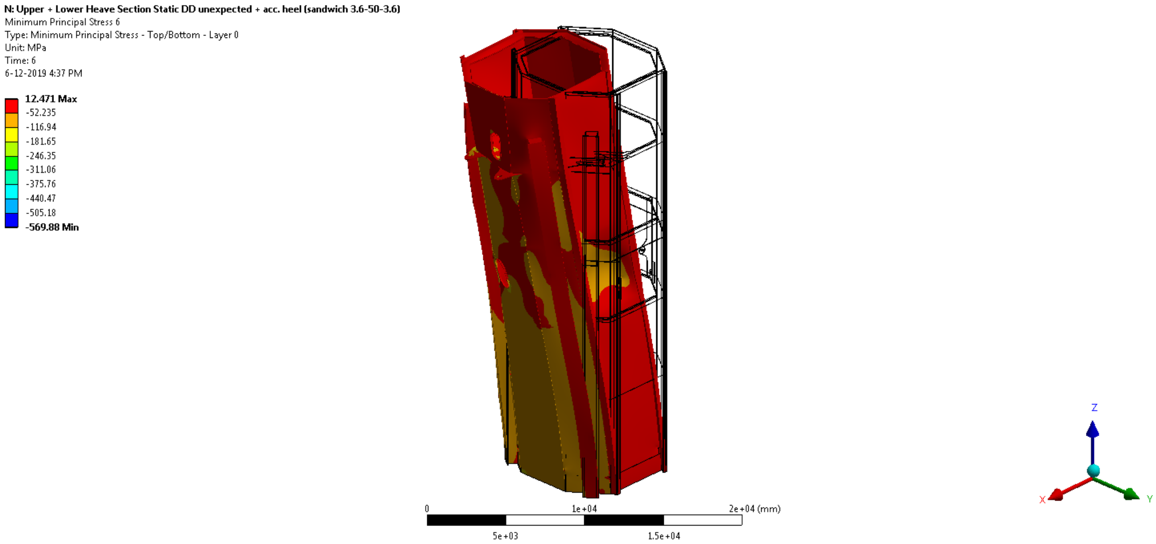


Front View

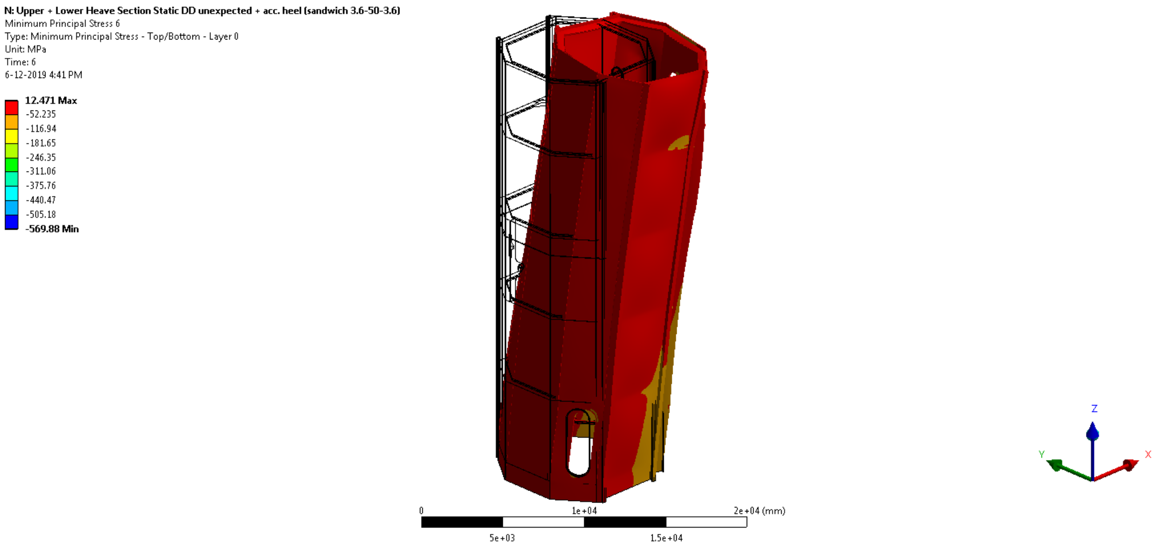


Back View

Figure F.17 Minimum Principle Stress (Load Case 5)

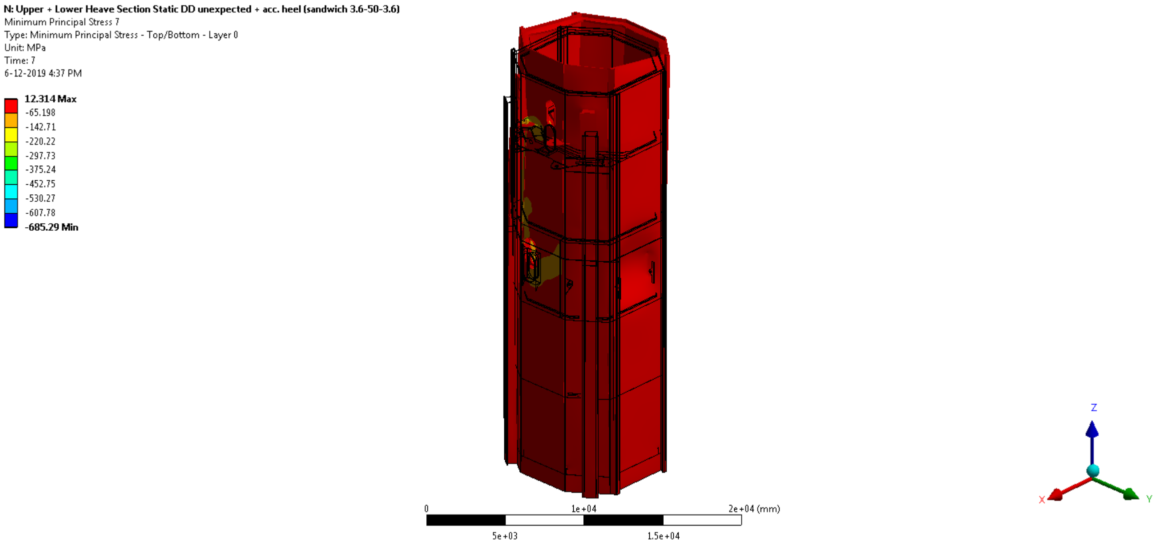


Front View

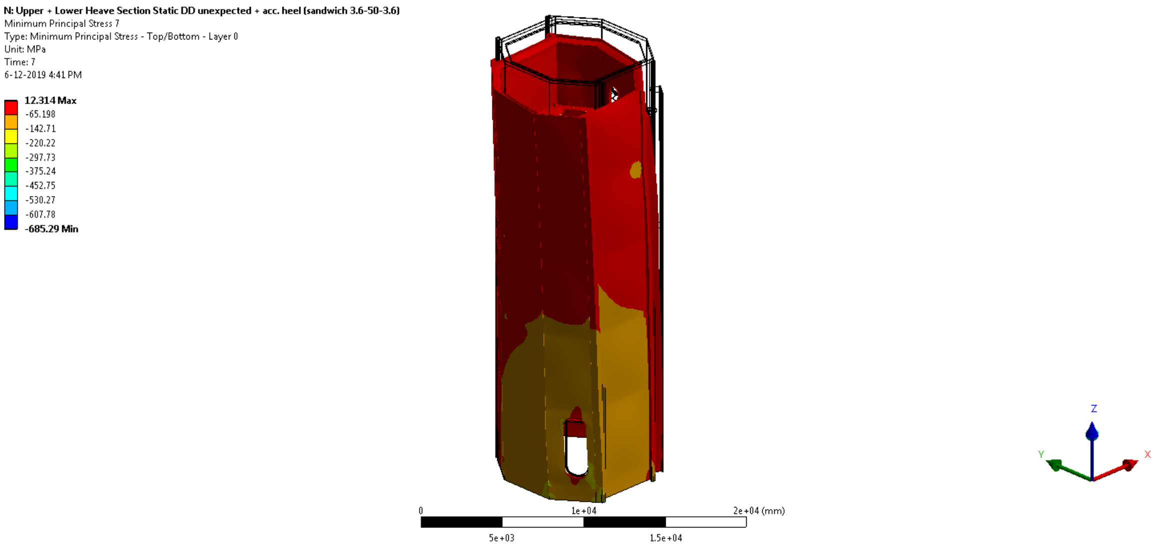


Back View

Figure F.18 Minimum Principle Stress (Load Case 6)

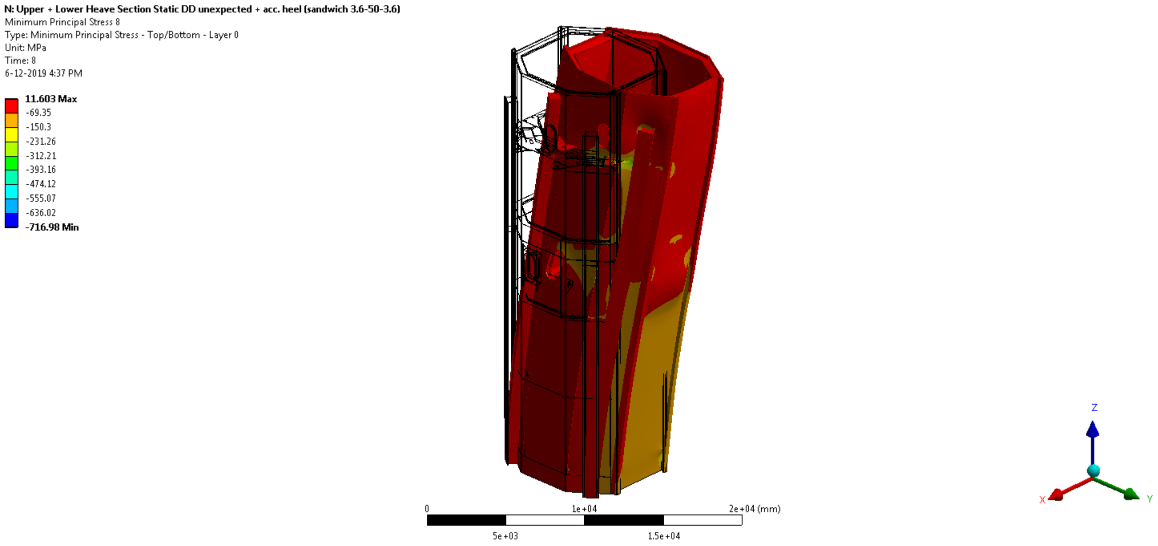


Front View

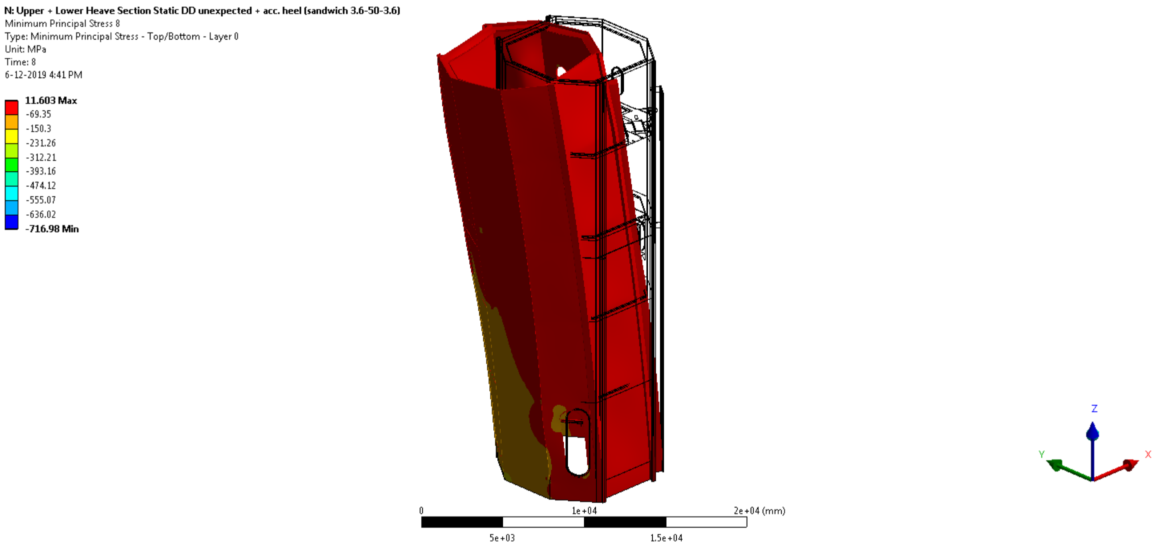


Back View

Figure F.19 Minimum Principle Stress (Load Case 7)

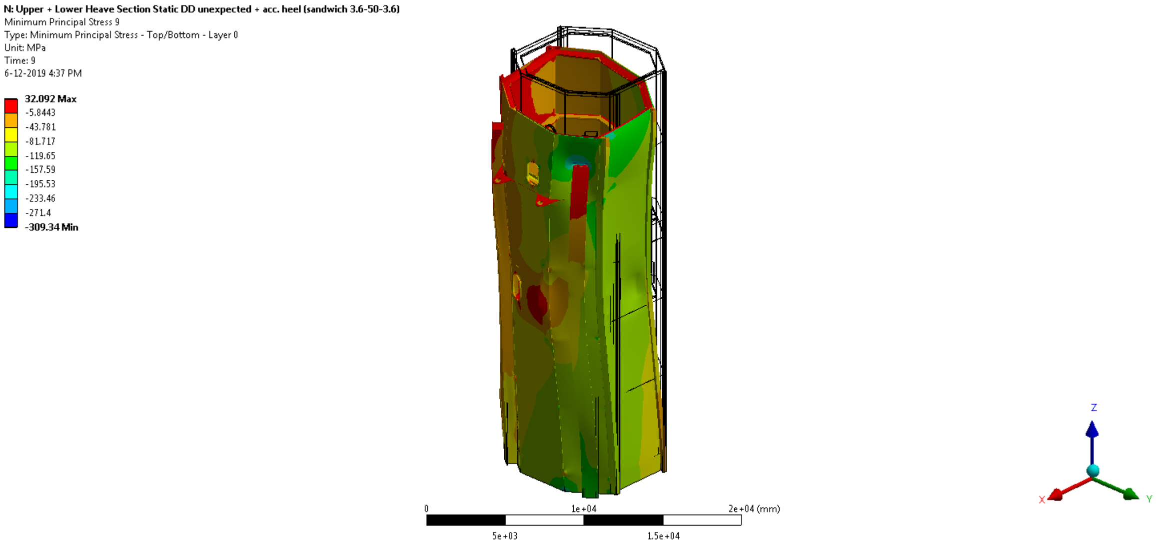


Front View



Back View

Figure F.20 Minimum Principle Stress (Load Case 8)



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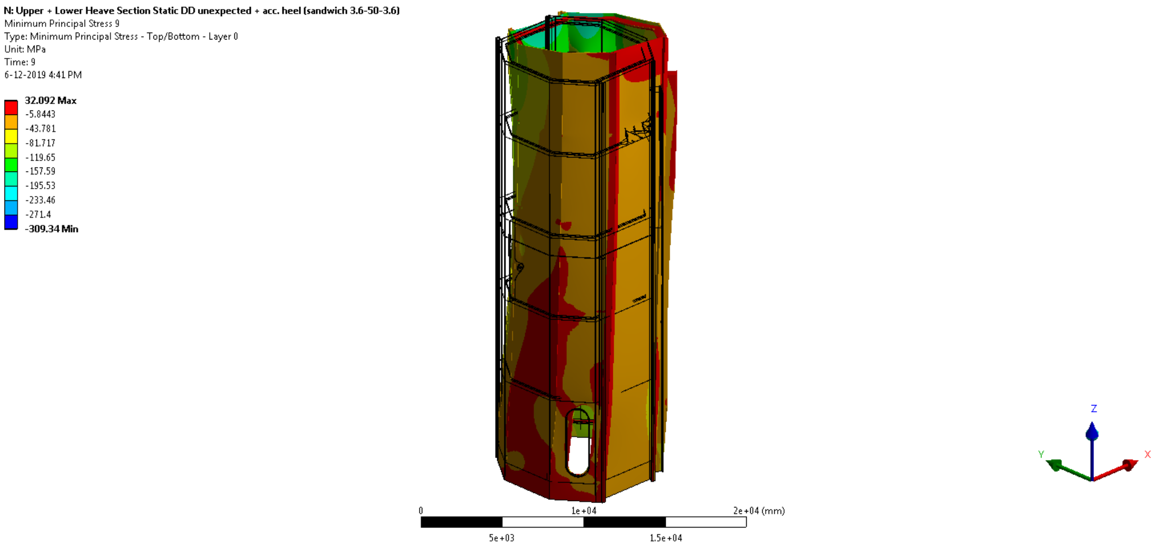
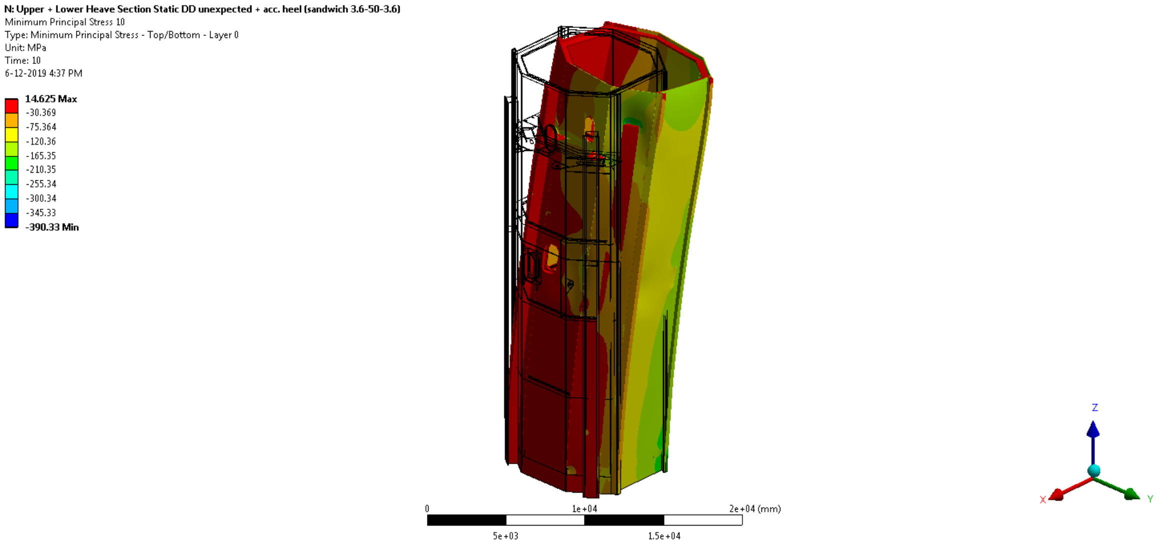


Back View

Figure F.21 Minimum Principle Stress (Load Case 9)



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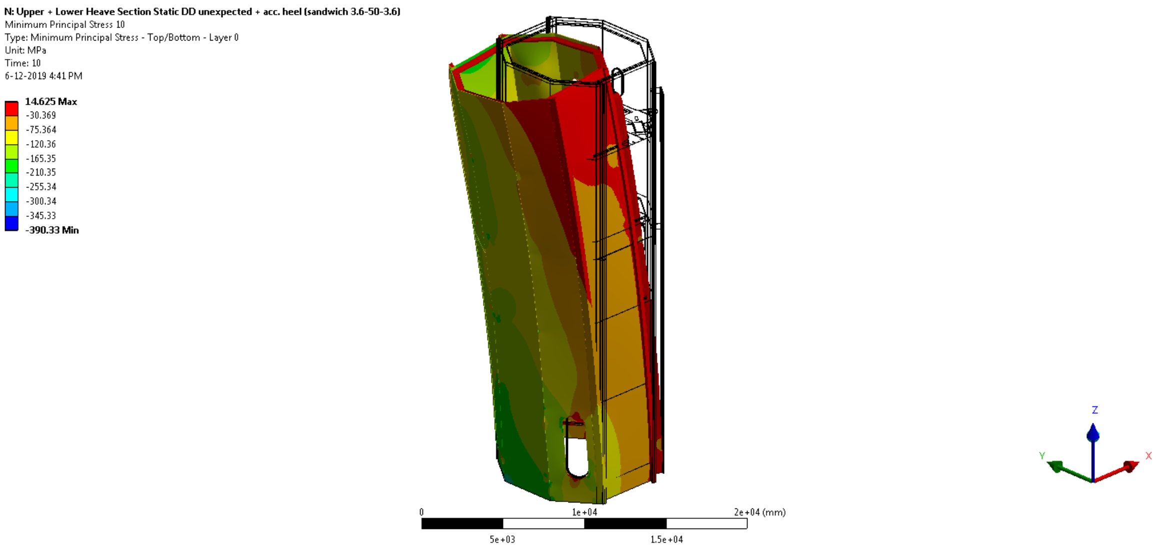
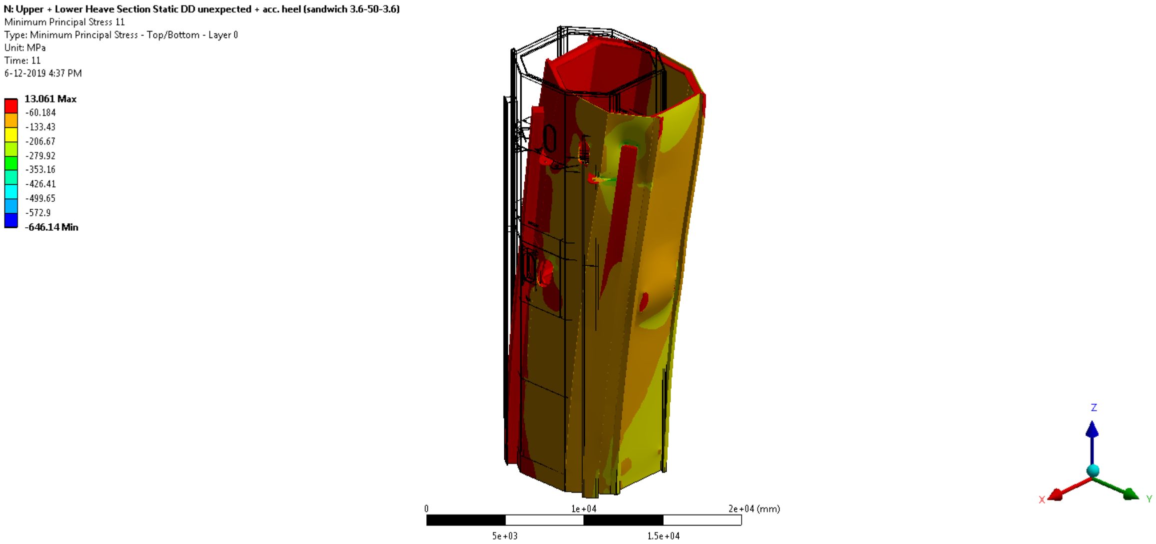


Back View

Figure F.22 Minimum Principle Stress (Load Case 10)



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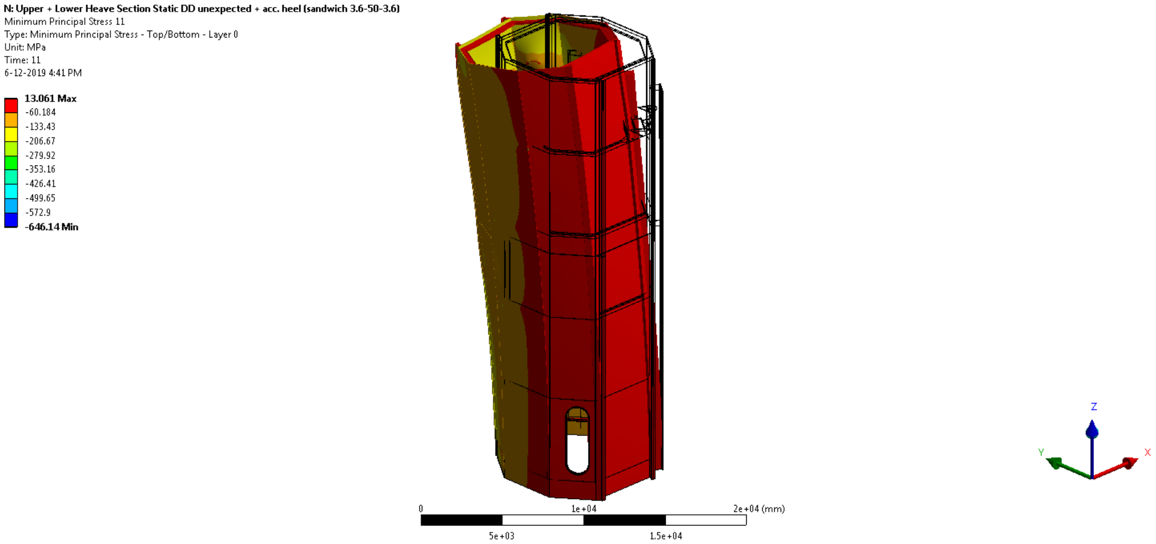
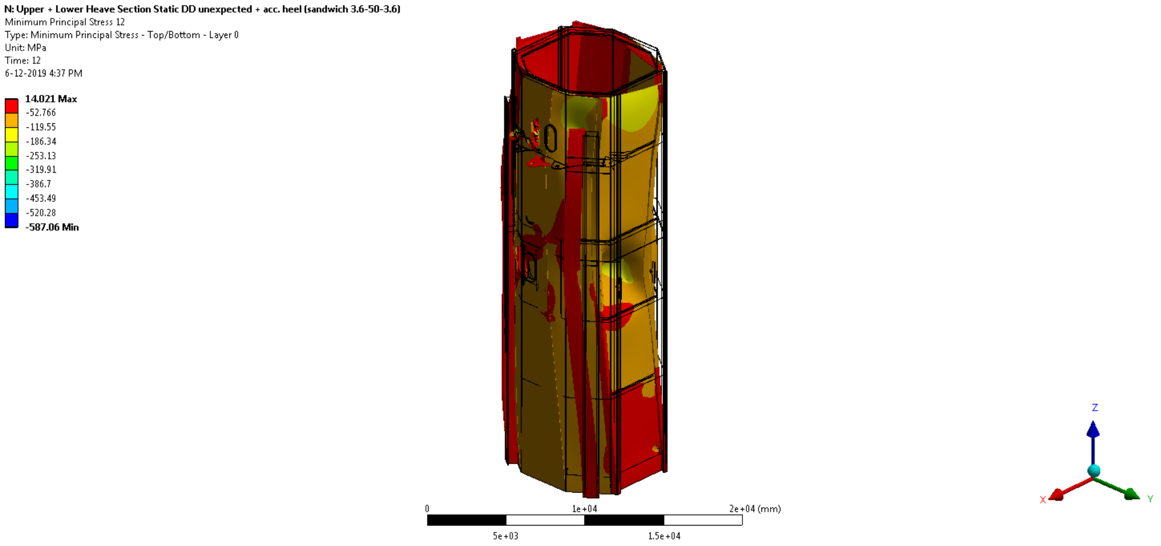


Back View

Figure F.23 Minimum Principle Stress (Load Case 11)



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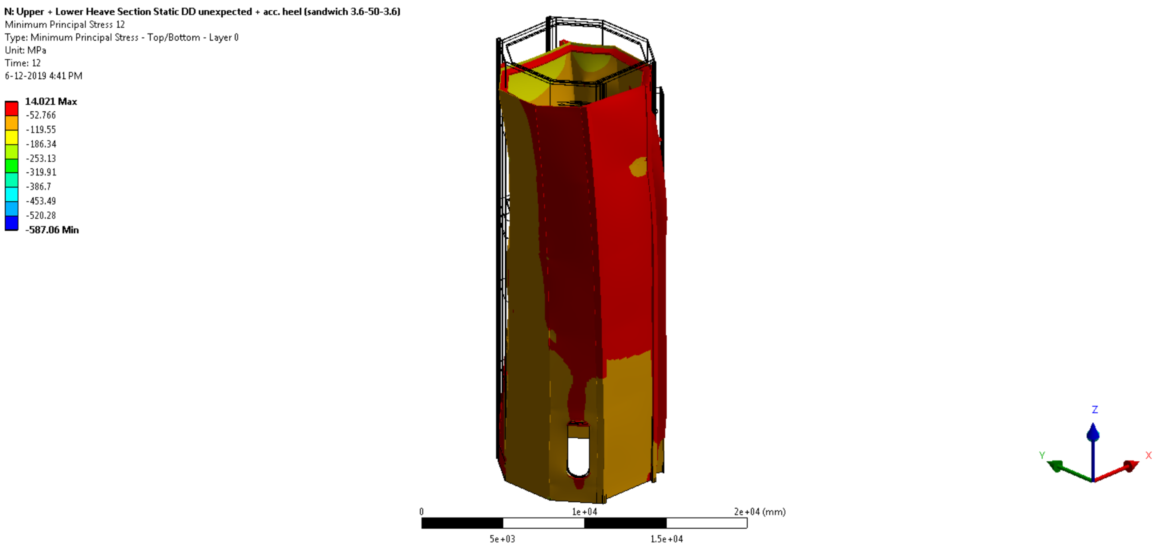
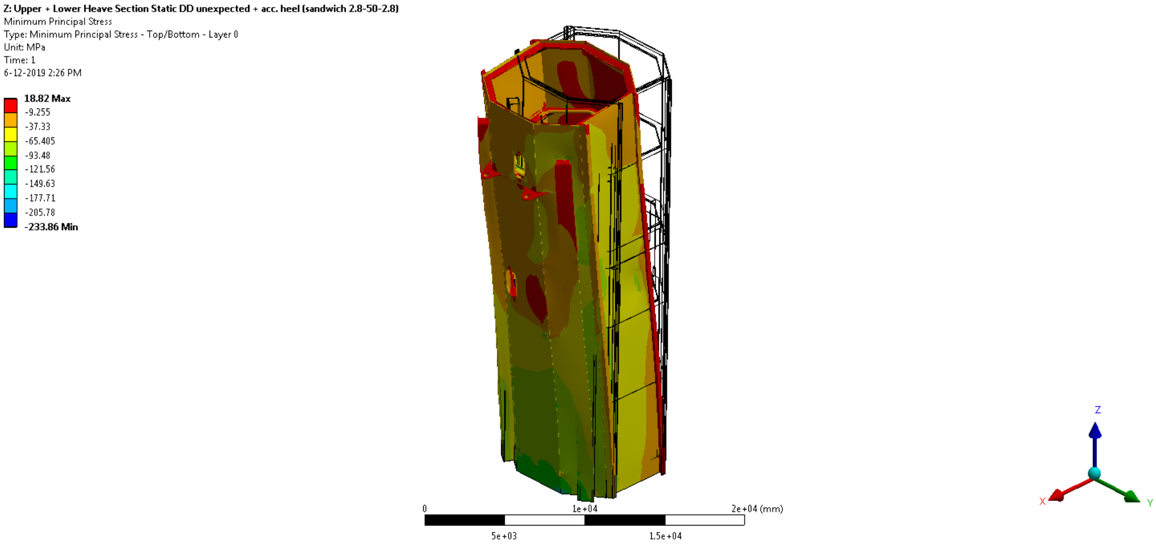


Back View

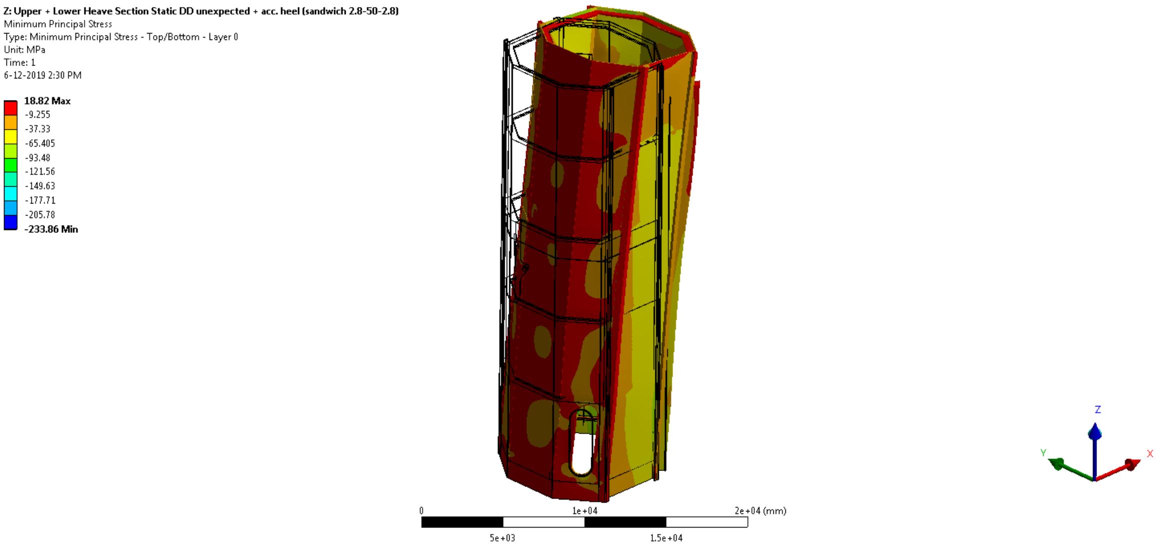
Figure F.24 Minimum Principle Stress (Load Case 12)

F.3 Sandwich with Faceplates S1100

The minimum principle stress generated in the structure due to different load cases is shown in the following figures.

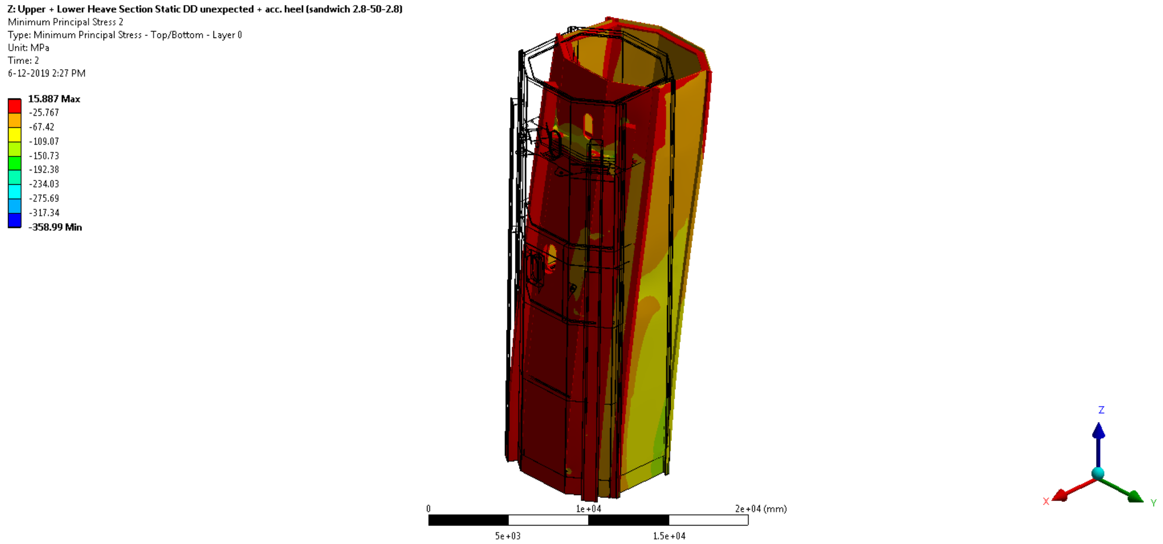


Front View

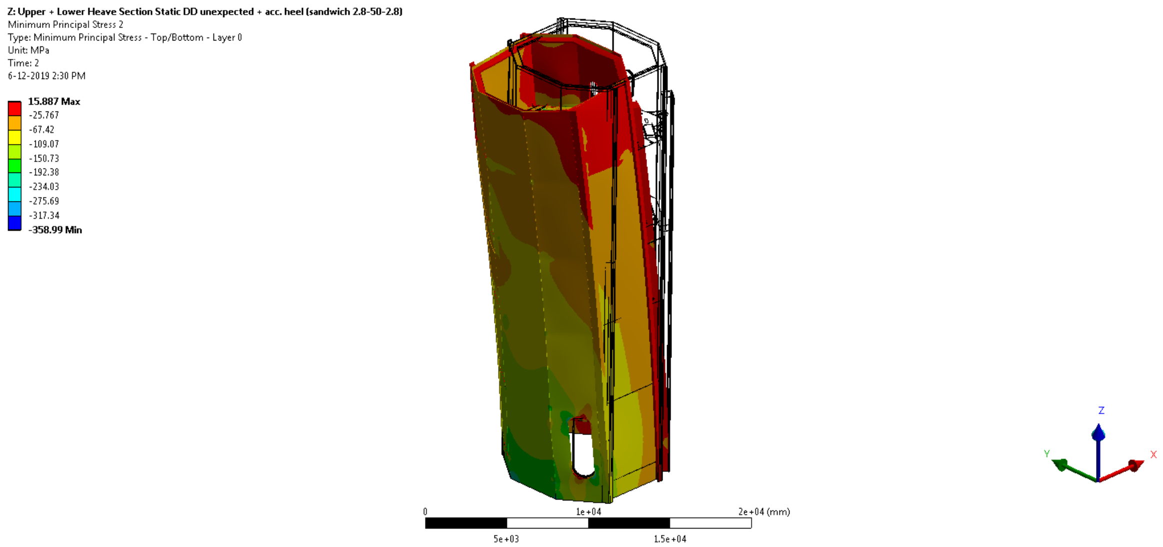


Back View

Figure F.25 Minimum Principle Stress (Load Case 1)

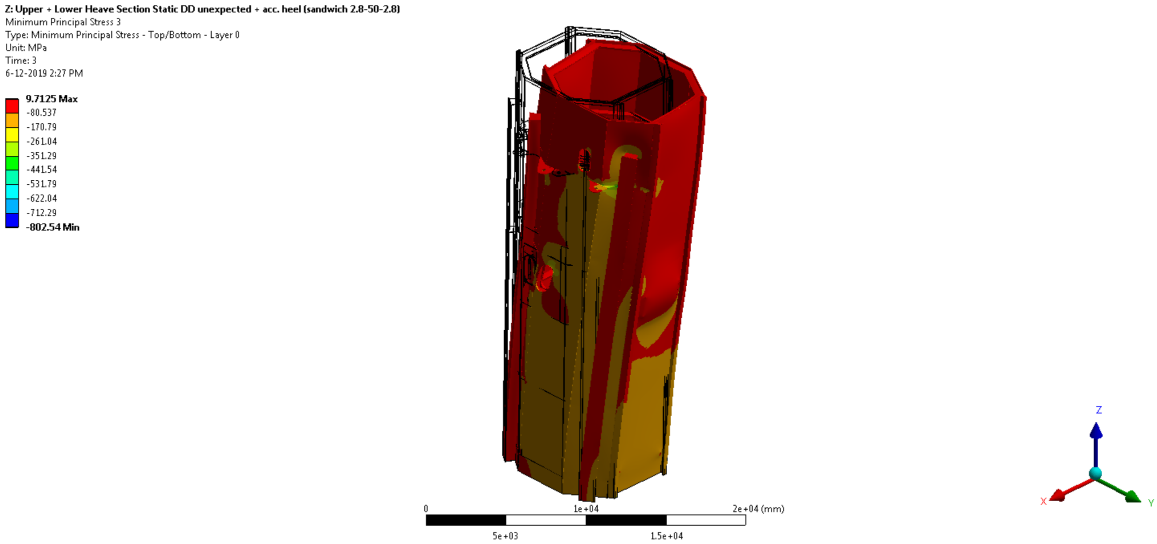


Front View

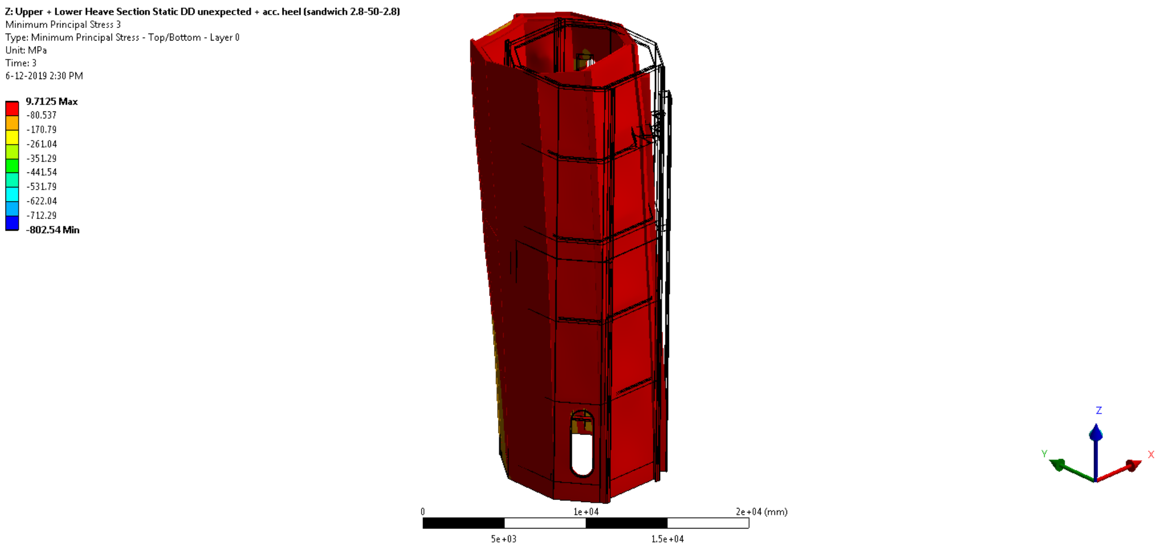


Back View

Figure F.26 Minimum Principle Stress (Load Case 2)

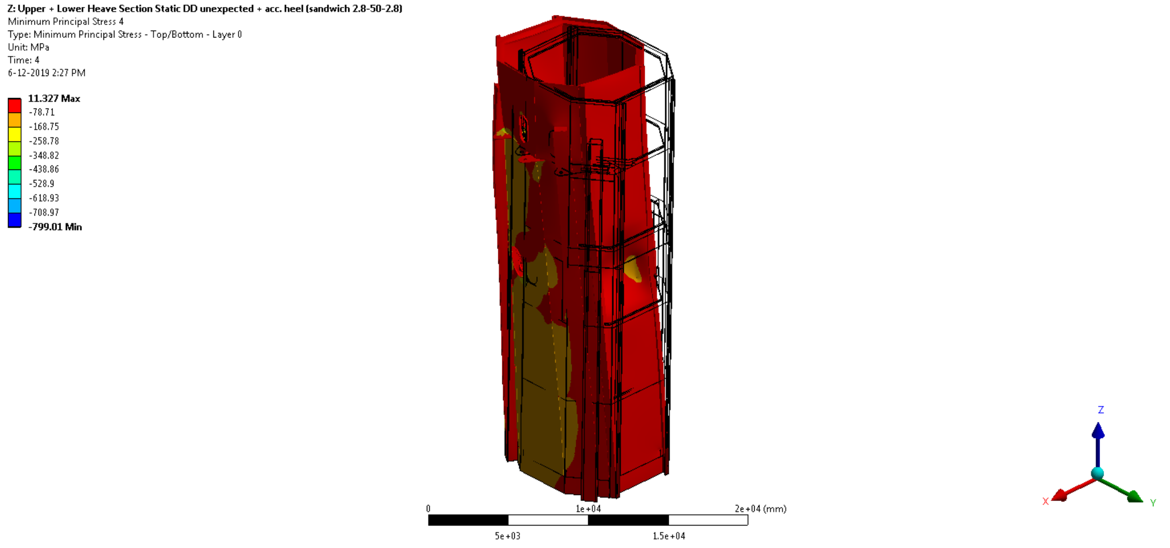


Front View



Back View

Figure F.27 Minimum Principle Stress (Load Case 3)



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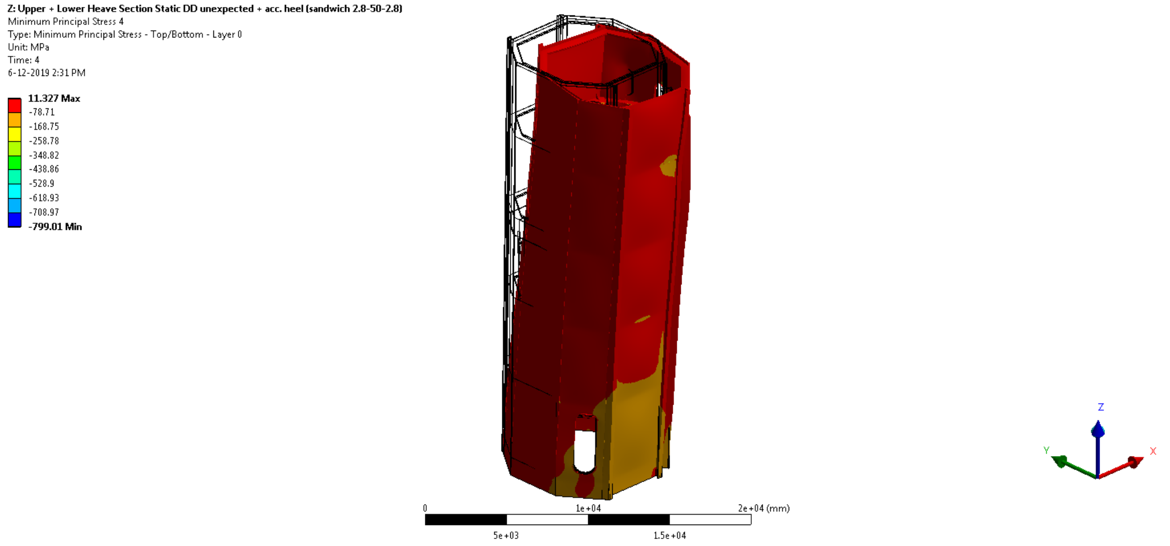
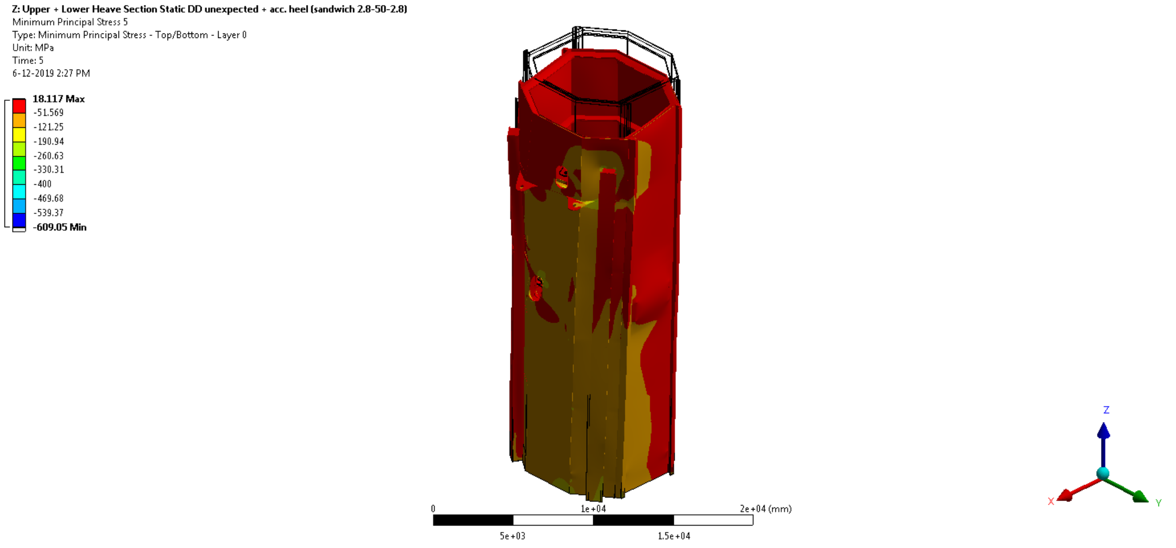


Back View

Figure F.28 Minimum Principle Stress (Load Case 4)



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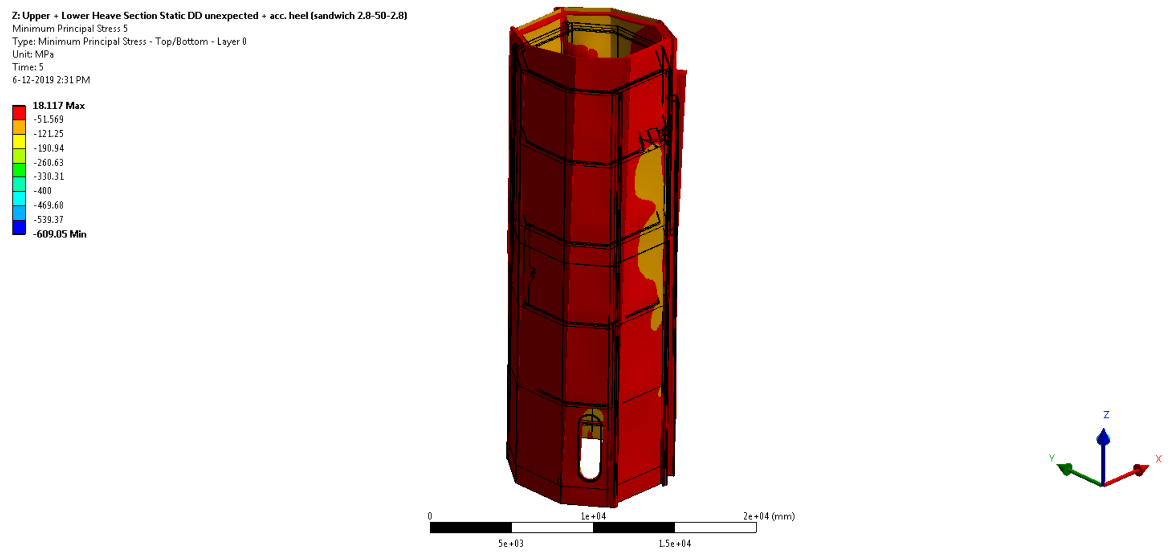
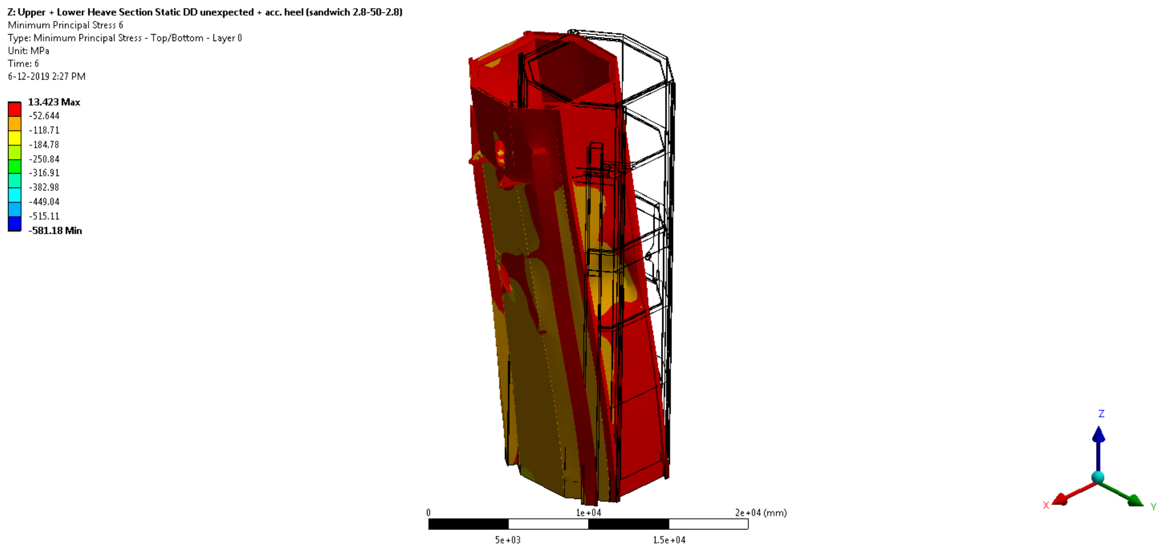
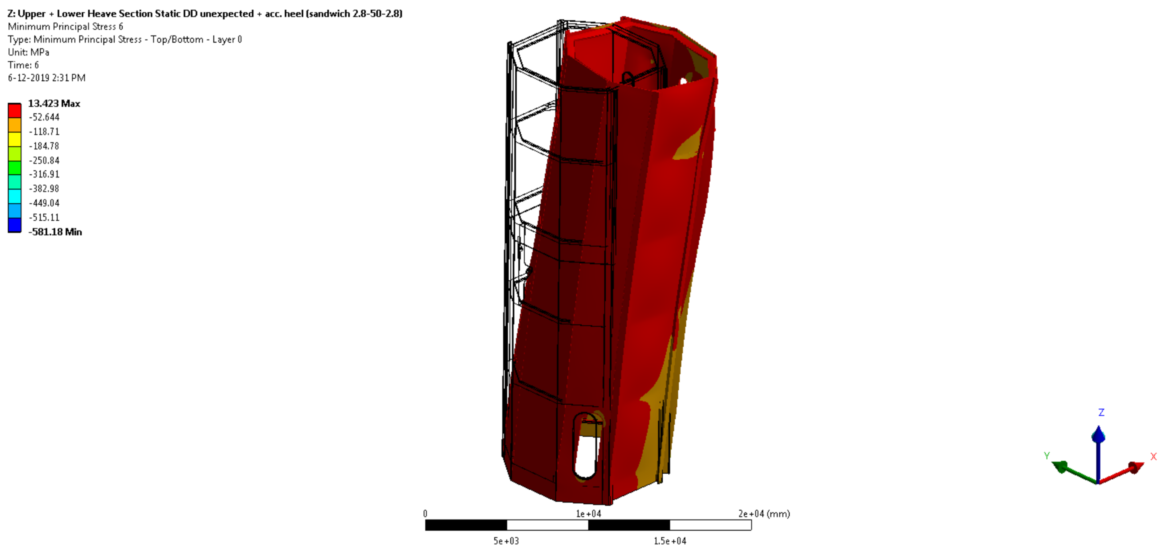


Back View

Figure F.29 Minimum Principle Stress (Load Case 5)

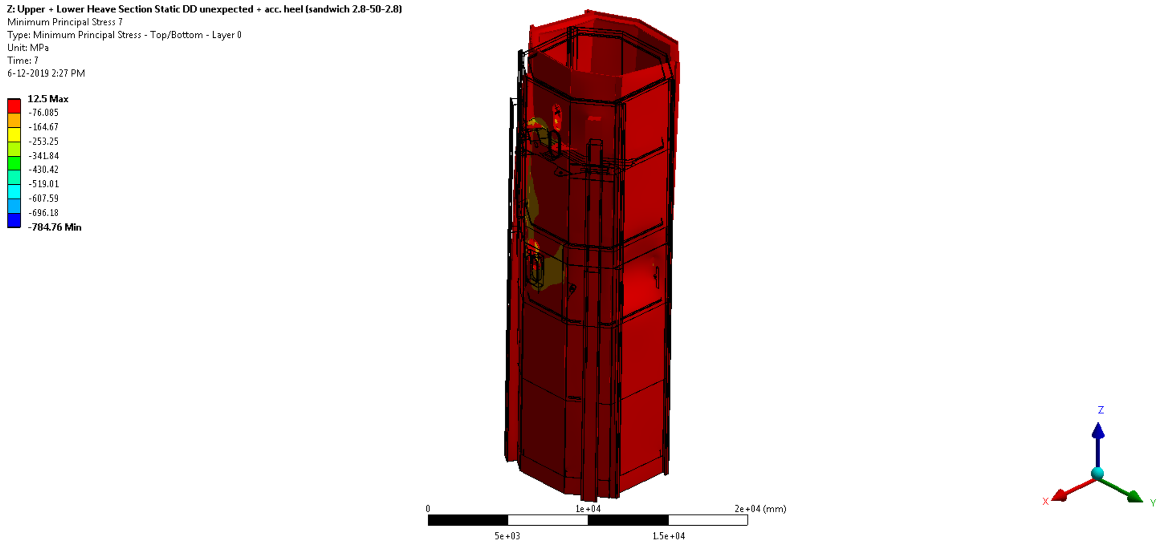


Front View



Back View

Figure F.30 Minimum Principle Stress (Load Case 6)



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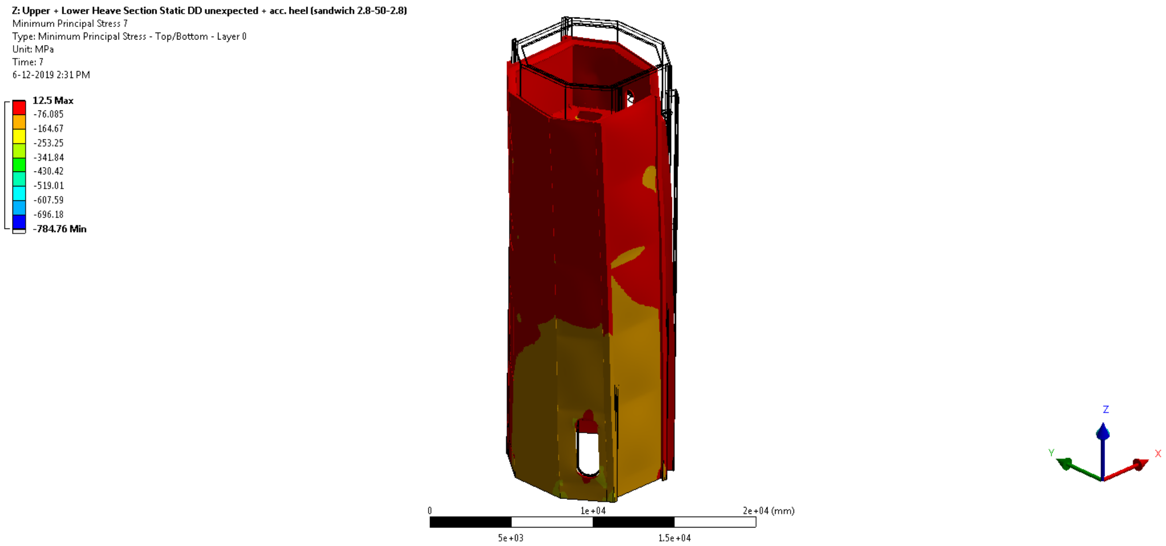
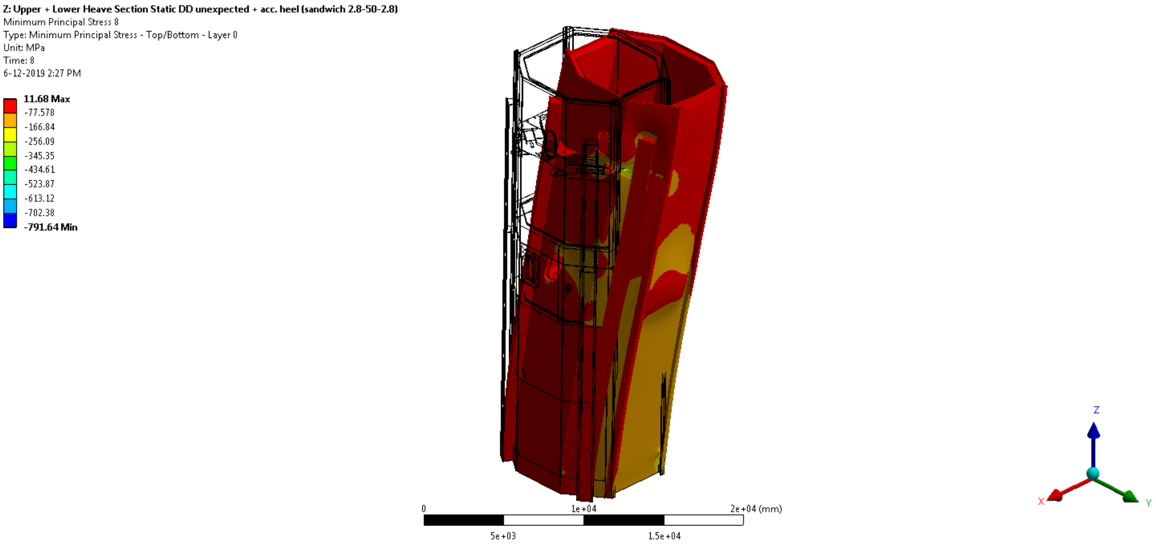
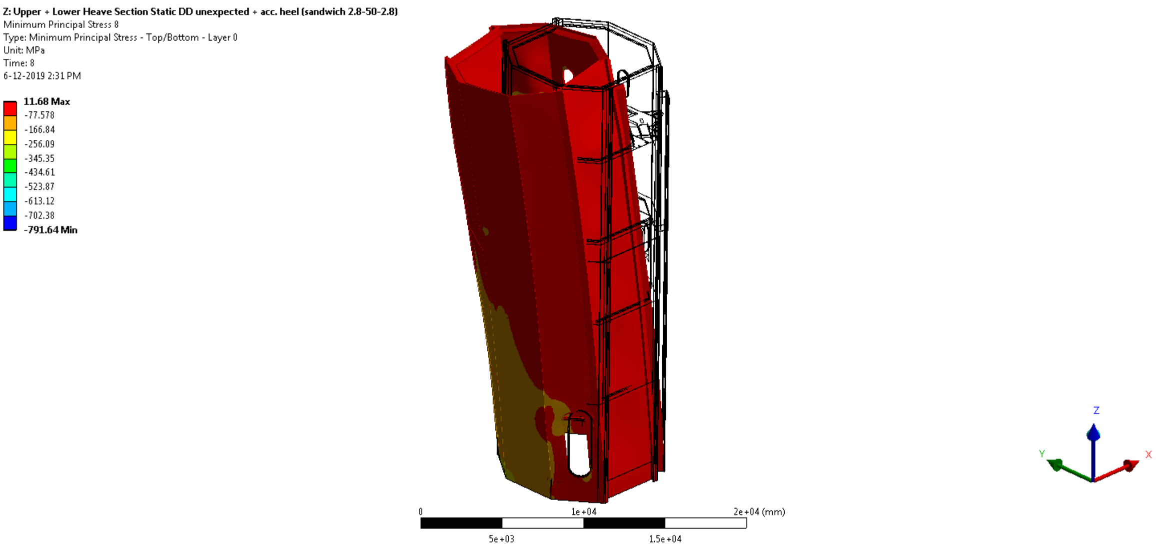


Back View

Figure F.31 Minimum Principle Stress (Load Case 7)

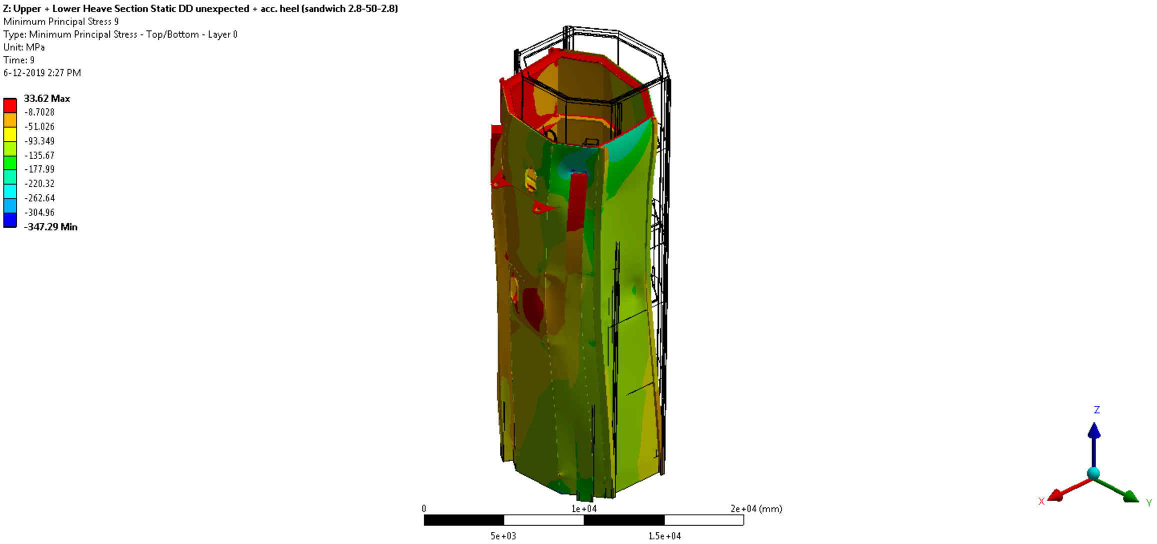


Front View

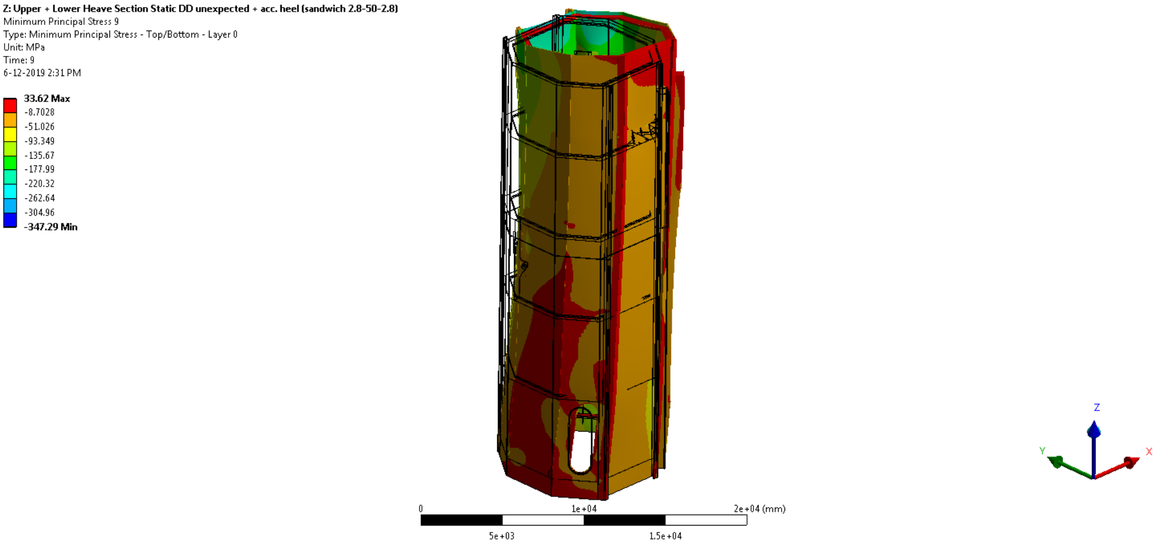


Back View

Figure F.32 Minimum Principle Stress (Load Case 8)

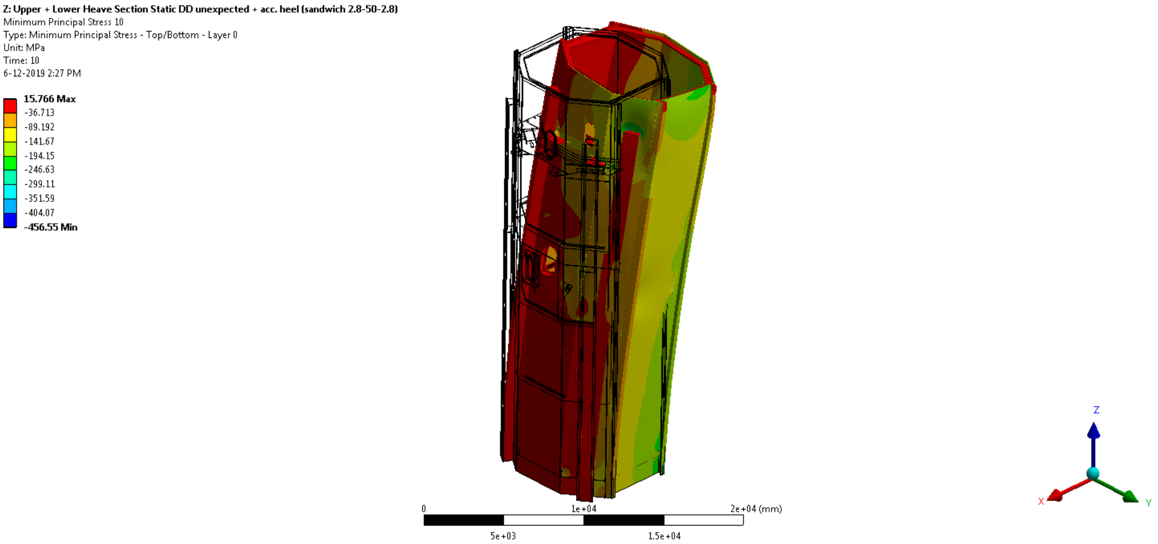


Front View



Back View

Figure F.33 Minimum Principle Stress (Load Case 9)



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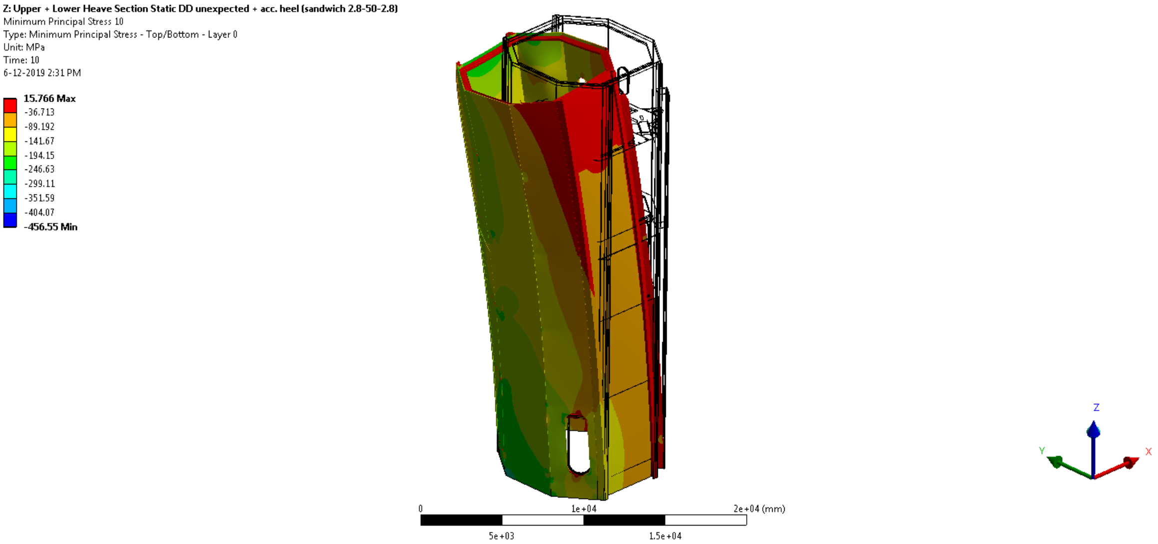
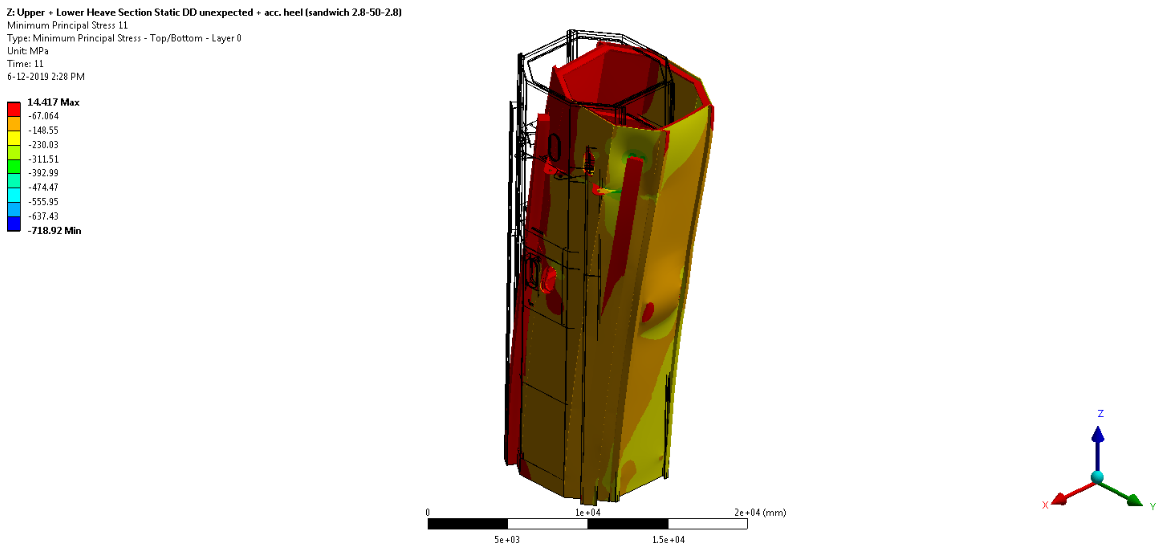


Back View

Figure F.34 Minimum Principle Stress (Load Case 10)



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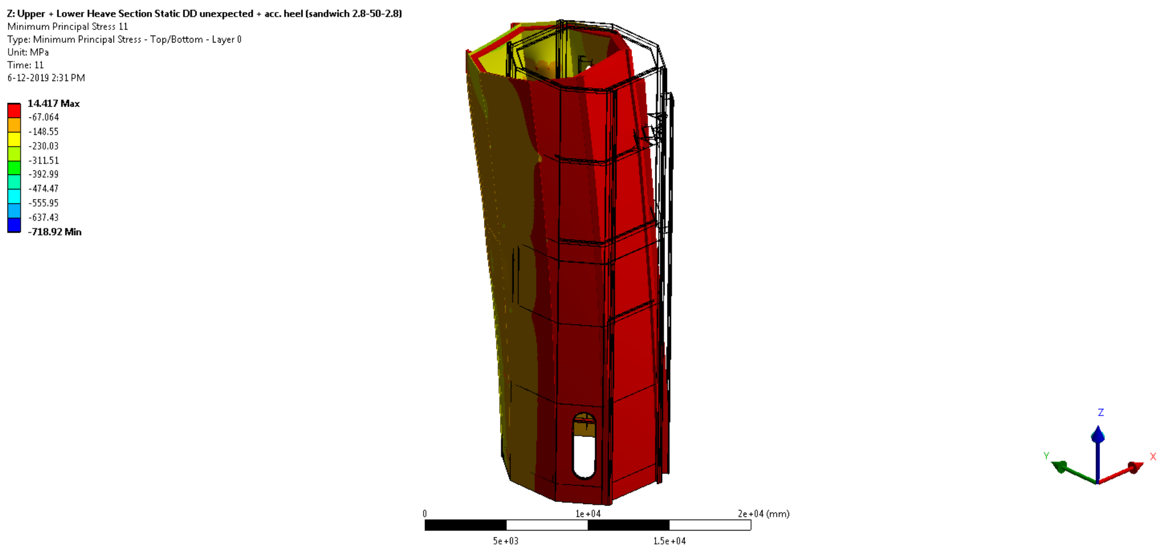
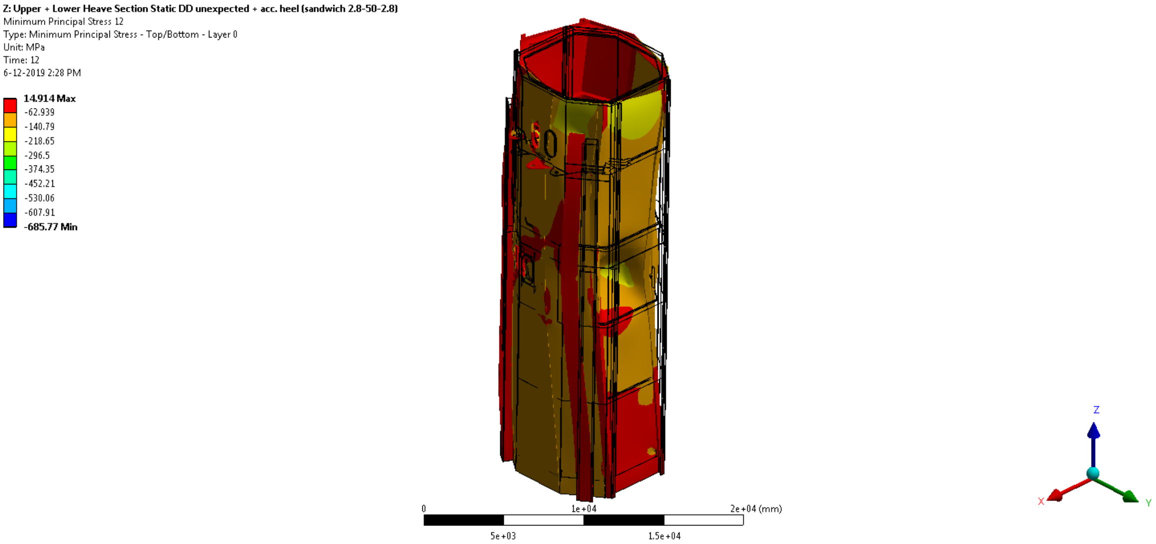


Back View

Figure F.35 Minimum Principle Stress (Load Case 11)



Front View

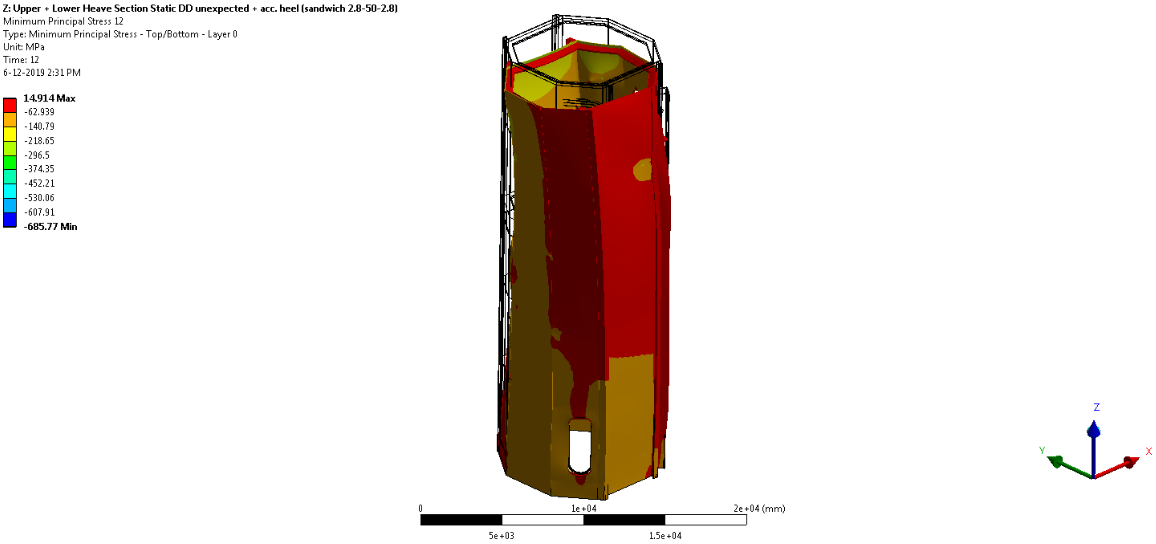


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Figure F.36 Minimum Principle Stress (Load Case 12)

Appendix G Comparison of Optimised Stiffened Plate and Sandwich Panel

Optimisation of structure is concerned with maximizing the utility of a fixed quantity of resources to full-fill the desired objective. The principle behind structural optimisation is minimum use of material for maximum performance. Structural topology optimisation is the most general type of optimisation technics. In topology optimisation, techniques can be applied to generalised problems with the use of finite element analysis. In the field of Aerospace, automotive and mechanical engineering, topology optimisation plays an important role in the design of lightweight and cost-effective structures. In an era of sustainable and resilient infrastructures, where the concept of redundancy plays a significant role, we should consider optimising every single structure to best of its efficiency.

G.1 Optimized Stiffened Plate

The stiffened plate used in Huisman structure is optimized from a practical sense. It was designed based on practical limitations and functional requirement of structure. But if practical limitations are ignored then by changing some dimensions and parameters it can be optimized from the theoretical point of view.

Basic dimensions i.e. length and width of the stiffened plate are kept constant, and thickness of the base plate and height/depth & thickness of stiffener can be changed so as to optimize stiffened plate. The stiffened plate is said to be optimized when critical stress is close to yield strength of the material used so that we can achieve maximum utilization of material. According to this, following dimensions of the stiffened plate are considered. The stiffened plate is shown in the following Figure G.1,

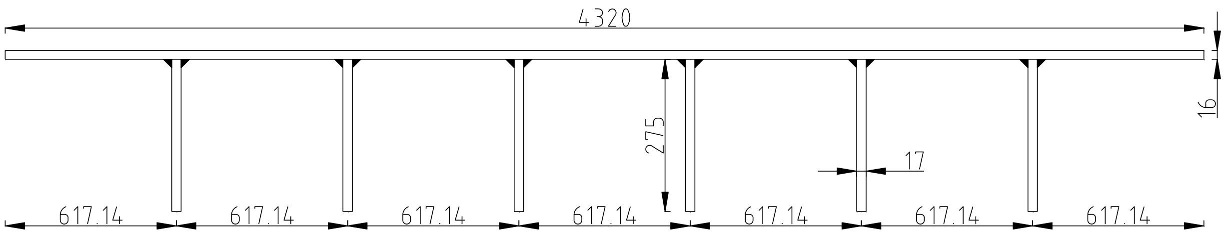

Figure G.2 Cross-section of stiffened plate

Properties of the stiffened plate are calculated and arranged in the following table,

Properties	*Stiffened Plate*
Critical Buckling Stress (MPa)	430
Buckling resistance (KN)	27200
Weight per unit length (gram/mm)	760
Total weight (KG)	4200

Table G.1 Properties of the stiffened plate.

Form this if we compare calculated critical stress and yield strength of the material, which is 355 MPa, we can observe that 87.8% of yield is used. Therefore, it can be concluded that the stiffened plate is pretty optimized.

G.2 Replacing Stiffened Plate by Sandwich Panel

There can be many configurations of a sandwich panel, which can be considered and evaluated to see if the same buckling resistance and weight reduction is achieved. But this is time consuming and long process. Therefore, the relationship between the thickness of core and faceplate can be found out, to achieve the same buckling resistance and reduced weight, as stated in the previous chapter. From this relation, the desired sandwich configuration can be found out and used for replacement of stiffened plate. If both graphs are combined, the range of thickness of core and thickness of faceplates can found out with which both objectives reduced weight and same resistance are achieved. In this chapter, three different sandwich panels will be considered with different yield namely S355, S690 and S1100. Buckling resistance and weight of these panels will be calculated and compared with stiffened plate, to evaluate if the selected sandwich gives the same buckling resistance and weight reduction at the same time.

G.2.1 Sandwich with Faceplates S355

Sandwich 9.5-70-9.5

Consider sandwich with faceplates S355. Two graphs are plotted to see the relationship between the faceplate and core thickness as stated before. These graphs are combined together as shown in Figure G.2. In graph brown line represent the relation between the thickness of core and faceplate to achieve the same resistance whereas the blue line represents the relation between the thickness of core and faceplate for buckling.

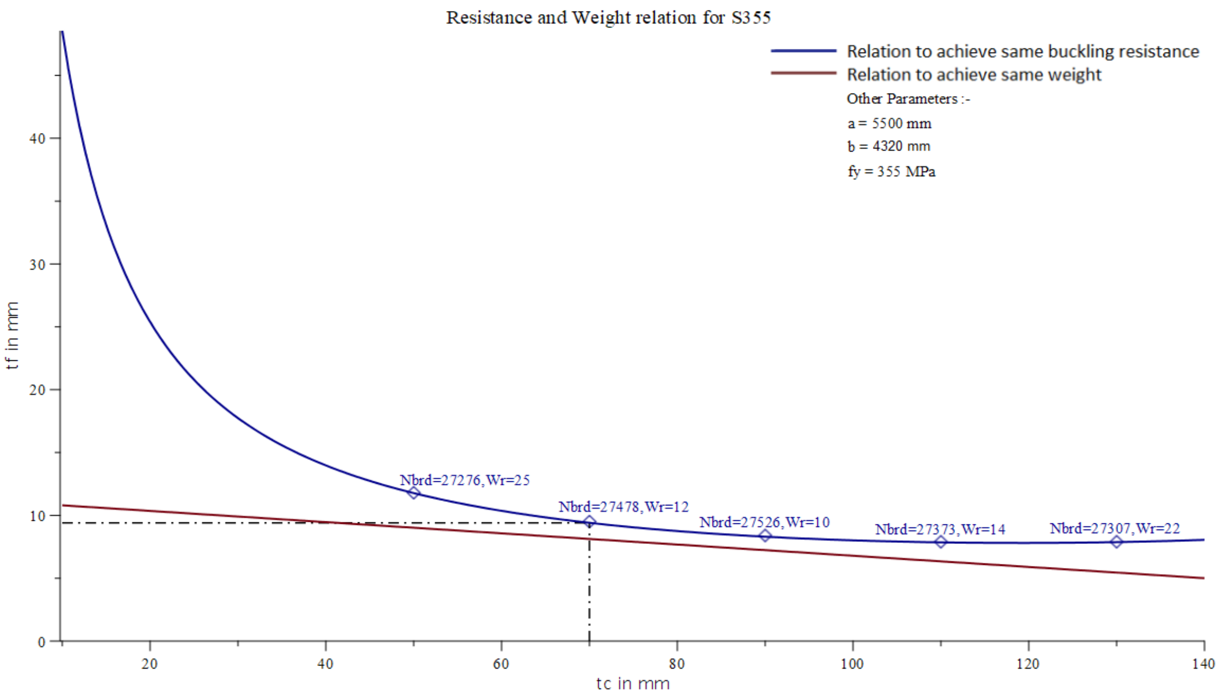

Figure G.3 Relation between the thickness of faceplate and core for buckling resistance and weight

From the above graph, it can be observed that a sandwich with faceplates S355 cannot fulfil both criteria simultaneously. Configuration of the sandwich to achieve the same buckling strength as that of the stiffened plate will have more weight than that of the stiffened plate. For example consider sandwich 9.5-70-9.5, buckling resistance of the sandwich panel is 27478 KN and weight 4708 KG. It has 12.2% more weight than the stiffened plate. Therefore, sandwich 9.5-70-9.5 cannot be used to replace the stiffened plate to achieve weight reduction.

Considered sandwich with core thickness 50 mm and faceplate thickness 5.4 mm is shown in the following Figure G.3.

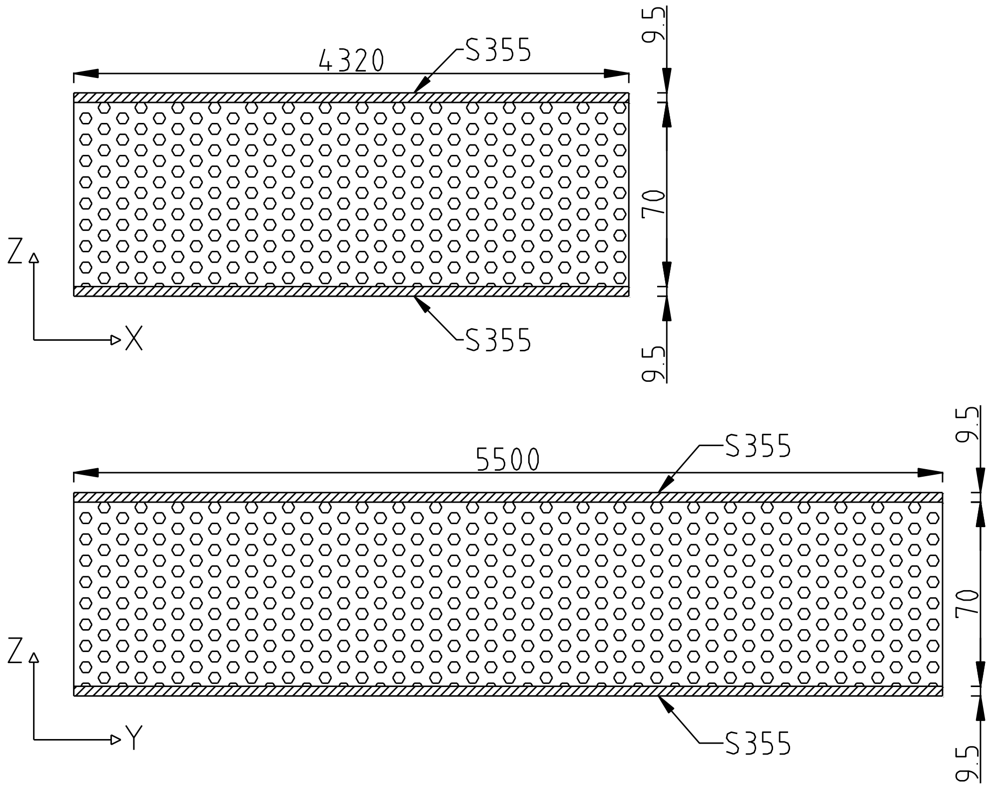

Figure G.4 Sandwich Panel 9.5-70-9.5

Various properties of this sandwich are calculated and arranged in the following table.

Properties	*Sandwich 9.5-70-9.5*
Critical Buckling Stress (MPa)	630
Buckling resistance (KN)	27500
Critical stress Core Shear (MPa)	710
Critical stress Face Wrinkling (MPa)	3200

Table G.2 Properties of sandwich 9.5-70-9.5

Fields of Comparison	*Stiffened Plate*	*Sandwich 9.5-70-9.5*
Critical Buckling Stress (MPa)	430	630
Buckling resistance (KN)	27200	27500

Table G.3 Comparison of between sandwich panel & stiffened plate

Fields of Comparison	Stiffened Plate	Sandwich 9.5-70-9.5
Weight per unit length (gram/mm)	760	860
Total weight (KG)	4200	4710
Weight reduction per unit length (gram/mm)	-	+90
Weight reduction for panel (KG)	-	+510
Percent weight reduction	-	+12

Table G.4 Comparison between the sandwich panel and stiffened plate

If sandwich 9.5-70-9.5 with faceplates of grade S355 is used then we can achieve buckling strength more than that of the stiffened plate. However, with the sandwich configuration that self-weight of the structure is increased. Increase in self-weight is 510 KG as compared to a stiffened plate which is 12 % of the stiffened plate. So the criterion of weight reduction is not achieved.

G.2.2 Sandwich with Faceplates S690

Sandwich 6.1-70-6.1

Consider sandwich with faceplates S690. Two graphs are plotted to see the relation between the faceplate thickness and core thickness as stated before. These graphs are combined together as shown in Figure G.4. In graph brown line represent the relation between the thickness of core and thickness of faceplate to achieve the same weight whereas green line represents the relation between the thickness of core and thickness of faceplate for buckling resistance.

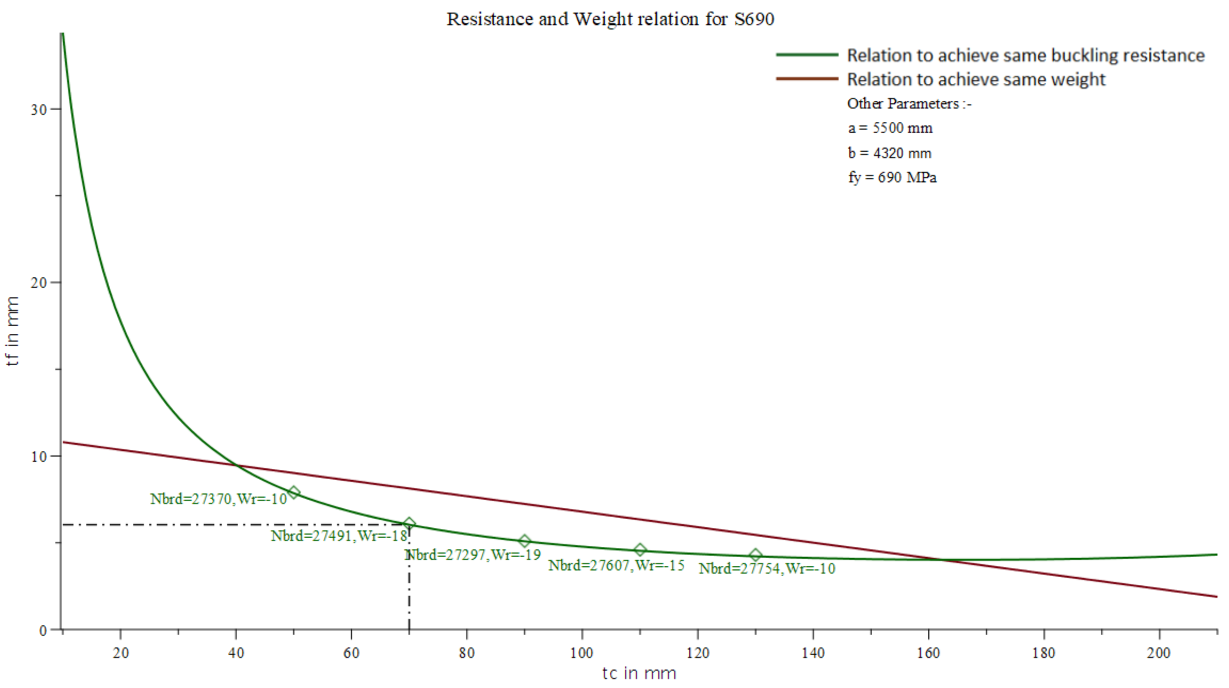

Figure G.5 Relation between the thickness of faceplate and core for buckling resistance and weight

From the above graph, we can predict a range of thickness of core and faceplate to achieve the same buckling resistance and lower weight as that of the stiffened plate. In the case where steel grade S690

is used for faceplates of sandwich panel, a range for the thickness of faceplate is between 4 mm to 10 mm and the corresponding range for core thickness is between 160 mm to 40 mm approximately. Consider a sandwich with a core thickness of 70 mm and faceplate thickness of 6.1 mm as shown in Figure G.5.

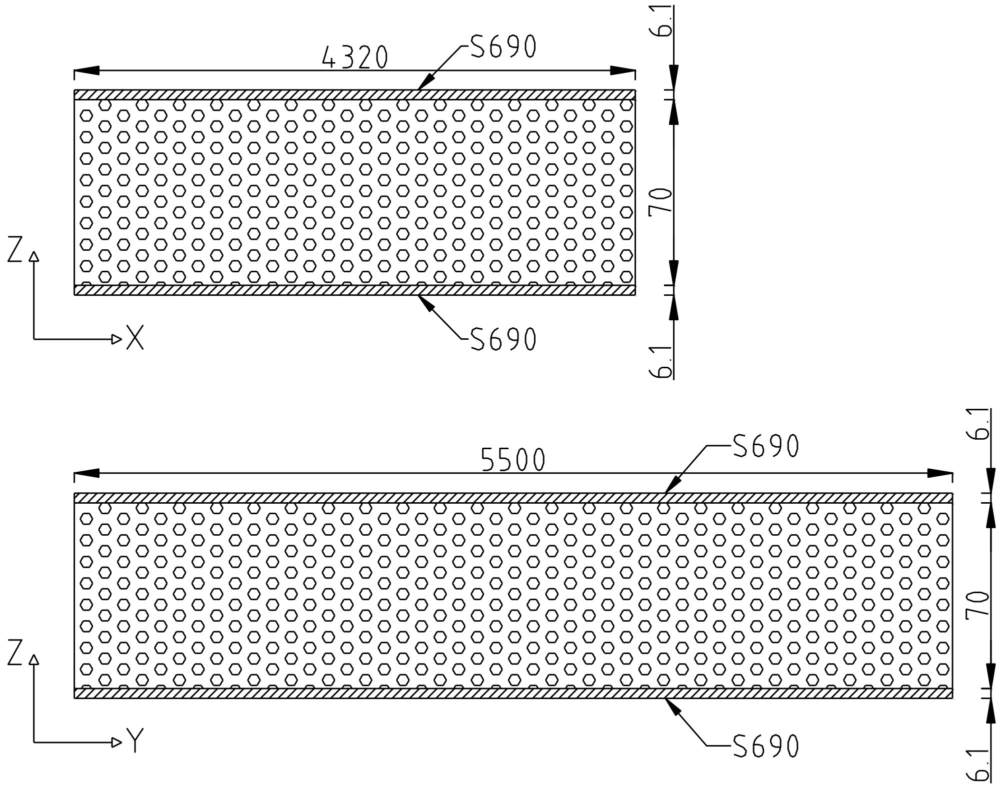

Figure G.6 Sandwich Panel 6.1-70-6.1

Various properties of this sandwich are calculated and arranged in the following table.

Properties	*Sandwich 6.1-70-6.1*
Critical Buckling Stress (MPa)	630
Buckling resistance (KN)	27500
Critical stress Core Shear (MPa)	1100
Critical stress Face Wrinkling (MPa)	2600

Table G.5 Properties of sandwich 6.1-70-6.1

Fields of Comparison	*Stiffened Plate*	*Sandwich 6.1-70-6.1*
Critical Buckling Stress (MPa)	430	630
Buckling resistance (KN)	27200	27500

Table G.6 Comparison of between sandwich panel & stiffened plate

Fields of Comparison	*Stiffened Plate*	*Sandwich 6.1-70-6.1*
Weight per unit length (gram/mm)	760	630
Total weight (KG)	4200	3440
Weight reduction per unit length (gram/mm)	-	-140
Weight reduction for panel (KG)	-	-760
Percent weight reduction	-	-18

Table G.7 Comparison between the sandwich panel and stiffened plate

If sandwich 6.1-70-6.1 with faceplates of grade S690 is used then buckling strength more than that of the stiffened plate can be achieved. Also, with this sandwich configuration, it can be observed that self-weight of the structure is reduced. Reduction of self-weight is 760 kg as compared to the stiffened plate which is 18 % of the stiffened plate.

G.2.3 Sandwich with Faceplates S1100

Sandwich 4.6-70-4.6

Consider sandwich with faceplates S1100. Two graphs are plotted to see the relation between faceplate thickness and core thickness as stated before. These graphs are combined together as shown in Figure G.6. In graph brown line represent the relation between the thickness of core and thickness of faceplate to achieve the same weight whereas the green line represents the relation between the thickness of core and thickness of faceplate for buckling resistance.

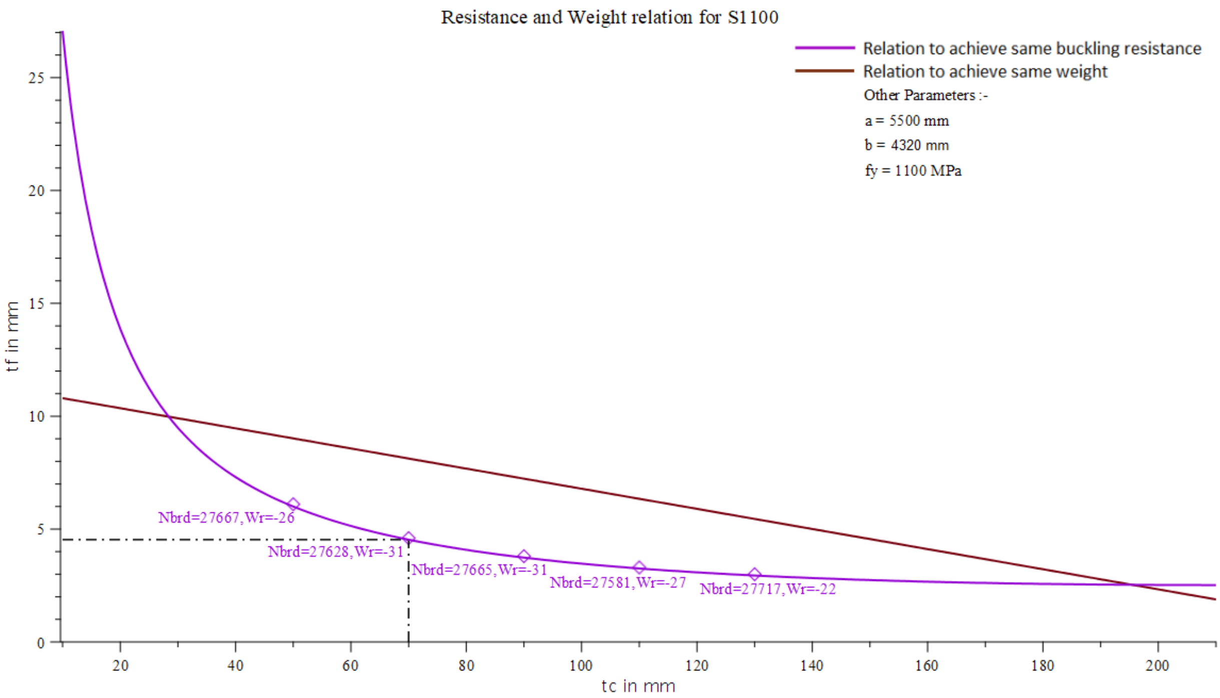

Figure G.7 Relation between the thickness of faceplate and core for buckling resistance and weight

From the above graph, we can predict a range of thickness of core and faceplate to achieve the same buckling resistance and lower weight as that of the stiffened plate. In the case where steel grade S1100

is used for faceplates of sandwich panel, a range for the thickness of faceplate is between 2.5 mm to 10 mm and corresponding range for core thickness is between 195 mm to 30 mm approximately.
Consider a sandwich with a core thickness of 70 mm and faceplate thickness of 4.6 mm as shown in the Figure G.8 below,

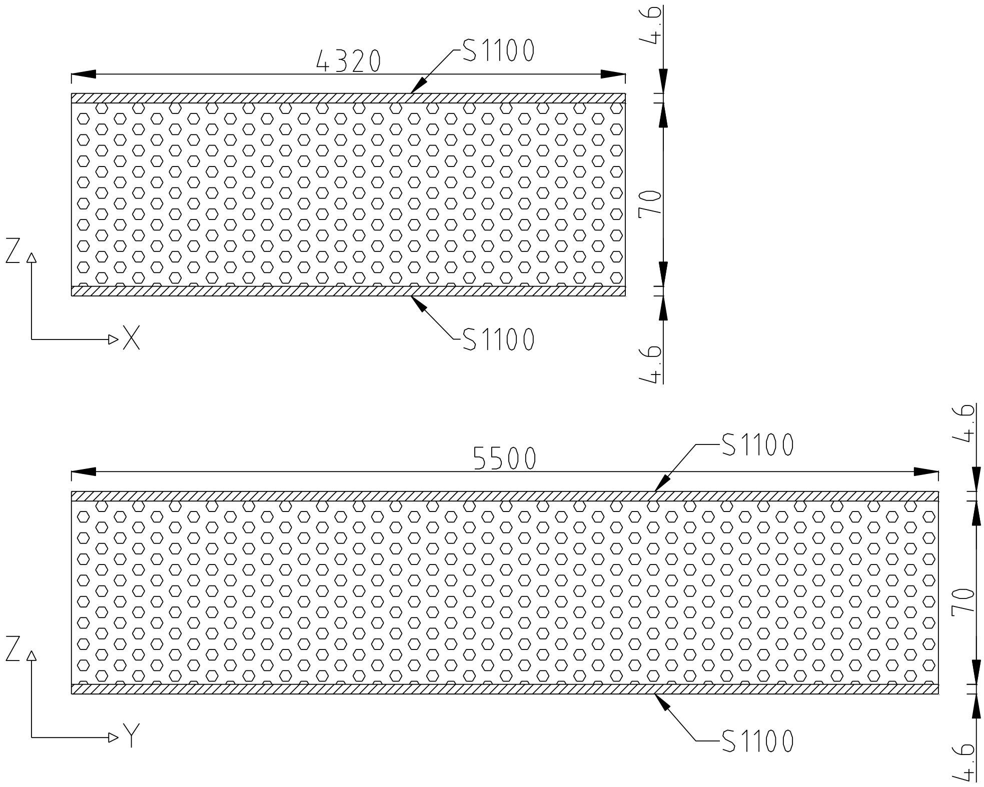

Figure G.9 Sandwich Panel 4.6-70-4.6

Various properties of this sandwich are calculated and arranged in the following table.

Properties	*Sandwich 4.6-70-4.6*
Critical Buckling Stress (MPa)	630
Buckling resistance (KN)	27600
Critical stress Core Shear (MPa)	1460
Critical stress Face Wrinkling (MPa)	2200

Table G.8 Properties of sandwich 4.6-70-4.6

Fields of Comparison	*Stiffened Plate*	*Sandwich 4.6-70-4.6*
Critical Buckling Stress (MPa)	430	630
Buckling resistance (KN)	27200	27600

Table G.9 Comparison of between sandwich panel & stiffened plate

Fields of Comparison	*Stiffened Plate*	*Sandwich 4.6-70-4.6*
Weight per unit length (gram/mm)	760	520
Total weight (KG)	4200	2880
Weight reduction per unit length (gram/mm)	-	-240
Weight reduction for panel (KG)	-	-1320
Percent weight reduction	-	-31

Table G.10 Comparison between the sandwich panel and stiffened plate

If sandwich 4.6-70-4.6 with faceplates of grade S1100 is used then buckling strength of the sandwich panel is more than stiffened plate. Also, with this sandwich configuration, it can be observed that self-weight of the structure is reduced. Reduction of self-weight is 1320 kg as compared to the stiffened plate which is 31% of the stiffened plate.

G.3 Result & Benefits

- It can be observed that with the use of a sandwich panel with high strength steel, the stiffened plate can be replaced.
- Use of the sandwich panel can also reduce the self-weight of structure.
- Use of extra high strength steel in the sandwich panel will reduce self-weight enormously. Also, there is a reduction in the thickness of the sandwich panel

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