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Redundancy Analysis during Progressive Buckling Collapse

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by

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ABSTRACT

The buckling failure of marine structures is an important aspect to be considered in structural design. One method to evaluate the integrity of the structure with buckling stresses is to evaluate the progressive failure of the structure. Progressive failure analysis of the panels involves identifying panels that are highly loaded in comparison to other panels and removing them over iterations and redistributing the loads to the remaining panels. This analysis is carried out using Numerical Methods, by developing various macros in ANSYS Mechanical APDL version 18.2 and a sample model. The midship section of a Cruise Vessel model is studied thoroughly, and the effect of the various loads on different parts of this selected section are analysed. The critical paths of failure of the different parts of the structure are identified. Parameter sensitivity analysis of the buckling resistance is also carried out. It is concluded that the considered structure is multiple redundant except in the event of extreme rare case scenarios. It is also demonstrated that when more than 70% of the panels of a structure fail at the first iteration, the overall structure fails at the end of the analysis. The various results obtained are relevant to the overall structural design and can be used to improve the design process of structures, however further improvements are required in order to improve the accuracy of the results.

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CONTENT

LIST OF FIGURES

LIST OF ABBREVIATIONS AND SYMBOLS

DNV-GL	Det Norske Veritas - Germanischer Lloyd
FEM	Finite Element Method
NFEM	Non-linear Finite Element Method
ISUM	Idealized Structural Unit Method
ULS	Ultimate Limit State
A	Cross Sectional Area
A_e	Effective Area
C_x	Buckling Factor for Stresses In X-Direction
C_{xs}	Effective Width Factor Due to Stresses In X-Direction
C_{ys}	Effective Width Factor Due to Stresses In Y-Direction
E	Young's modulus of elasticity
I	Moment of Inertia
M_{Rd}	Design Bending Moment Resistance
M_{Sd}	Design Bending Moment
c_i	Interaction Factor
f_{cr}	Elastic Plate Buckling Strength
f_d	Design Yield Strength
f_E	Euler buckling strength
f_k	Characteristic Buckling Strength
l_e	Effective Length
s	Plate Width, Stiffener Spacing
s_e	Effective Width of Stiffened Plate
λ_p	Reduced Plate Slenderness
$\sigma_{x,Rd}$	Design Buckling Resistance under Longitudinal Compression Force
$\sigma_{y,Rd}$	Design Buckling Resistance under Transverse Compression Force
$\sigma_{j,Sd}$	Design Von Mises' Equivalent Stress
τ_{Rd}	Design Resistance Shear Stress
τ_{Sd}	Design Shear Stress

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1 INTRODUCTION

1.1 Background and Problem Formulation

The aim of a marine structural designer is to design structures capable of withstanding all the environmental challenges and its consequent loads. Buckling of structures is particularly a critical aspect in marine structural design, as the critical stress value in buckling is typically lesser than other loads. Buckling is, besides yielding, one of the major causes of failures in structures. This results in marine structures being prone to buckling failure, and therefore the study of the buckling of different components of the structure is of importance when designing ships and offshore structures. Buckling typically happens on both a local and global scale – under different 'modes' of failure, which are detailed in this thesis. One approach of ensuring the structural integrity of structures is by designing the structure for redundancy. Redundancy of a component or a system is defined as the capability to keep up or reestablish its function when a failure of a member or connection has occurred. In the event of extreme loads – such has high sea-state wave loads, slamming loads – the vessel will be subject to extremely high-stress values. By designing the structure for redundancy, the integrity of the structure under these extreme loads can be evaluated. For estimating this is by evaluating the progressive failure of different components under these loads and verifying if the overall structure can bear these extreme loads even when a fraction of its components loses its load-bearing capacity.

Progressive collapse occurs when local structure damage causes a chain reaction of structural elements failures disproportionate to the initial damage resulting in a partial or full collapse of the structure. The potential progressive collapse analysis entails evaluating a structure for its vulnerability to the development of a partial or a total collapse of the structure initiated by an event that causes local damage. However, progressive collapse is difficult to totally avoid, therefore means of assessing the collapse of local structural members are needed, followed by a necessity of strengthening measures so as to have a redundant system. There are different methods to evaluate progressive collapses – most commonly used is Non-Linear Finite Element Method (NFEM). While FEM is a powerful tool for solving nonlinear structural problems, a weak feature of the conventional FEM is that it requires enormous modelling effort and computing time for nonlinear analysis of large-sized structures. However, a thorough investigation of the ship model by computing the stresses occurring in panels using finite element analysis is beneficial as the numerical model can help gain insight into possible structural failure due to buckling or yielding and a detailed view of the progressive critical path of the structural panels. As buckling is a critical aspect of vessel design, a detailed parameter sensitivity analysis of the structural buckling resistance under different loadings is carried out.

Numerical calculations are extremely time-consuming and commercial buckling check programs such as NFEM are expensive and carry out computationally intensive analysis. Therefore, there is a need for buckling check tools based on the recommendations of a classification society that are fast to use and easy to share. The author of the thesis believes that a simple tool that can evaluate panel failure and progressive collapse can be of great aid to designers when designing structures for buckling loads.

In this thesis, the progressive collapse of a section of the hull in the vicinity of the midship of a Cruise Vessel is studied in detail. The purpose of redundancy analysis for a cruise vessel is to prove that nevertheless the ship structure's panels are subjected to fail by yielding or buckling, due to loads forcing the material to exceed its yielding or ultimate strength criteria, it will be able to redistribute the loads to the remaining safe structural elements, providing structural integrity.

1.2 Structure of the Thesis

This thesis is divided into the following chapters:

Chapter 2: Theory
The general theory for stiffened plate buckling, local plate buckling, and progressive collapse are sequentially described and the failure modes of buckling are elaborated in this Chapter.

Chapter 3: Rule-Based Buckling Strength Analysis
The various expressions from DNV-GL RP-C-201 which details the buckling stresses and utilization factor are presented.

Chapter 4. Methodology for Numerical Modelling to Evaluate Progressive Structural Collapse

This chapter contains a detailed description of the methodology used to evaluate the progressive failure of the structure, along with the various programming macros used. The results from a sample model are also presented here.

Chapter 5. Results

The results of the analysis for the midship section of the Cruise Vessel – termed Main Model - is presented in this chapter, along with inferences from the various results obtained. The parameter sensitivity analysis of the utilization factor is also presented in this chapter.

Chapter 6. Conclusion

The conclusion of the present work and recommendations are also provided.

Chapter 7. Further Work

This chapter includes possible future improvements that can be carried out.

2 THEORY

2.1 Buckling and Typical Failure Modes

2.1.1 Buckling

Buckling is a phenomenon in which a structure loaded in compression or shear can experience deflections perpendicular to the plane of the applied load, and thereby fail. Ship structural members are subjected to a wide variety of hull girder and local loads during the vessel's operation among sea waves, and the degree or severity of failure may vary from a minor degradation of aesthetics to complete failure of the vessel.

A stiffened plate structure which has buckled can become unstable after deforming and thereby cannot perform its function. This decrease in load-carrying capacity of the collapsed panel may lead to the redistribution of stresses in the adjacent panels. The problem of stability of stiffened plate members is of great significance as disregarding it may lead to disastrous results such as the subsequent collapse of the entire structure as shown in Figure 1.

A major requirement for any marine structure is to have low initial and operational costs, to be reasonably safe, to not have catastrophic failure nor to have much trouble in service due to frequent minor failures. The vessel should be designed to have adequate strength to resist the loads as the overall safety is concerned not only with the structure itself but also with external damage that may result as a consequence of failure.

Figure 1. Local structural collapse/plastic buckling/damages in ship structures (DNVGL, 2015)

Elastic Buckling

Elastic buckling refers to when the structure returns to its original undeformed condition once the compressive load is removed. In the case of flat plate buckling, the elastic buckling stress is the highest value of the compressive stress in the plane of the initially flat plate, in which a nonzero out-of-plane deflection of the middle portion of the plate can exist. A plate in compression will have a critical buckling load and exceeding this load by a small margin will not necessarily result in a complete collapse of the plate but only in an elastic deflection of the central portion of the plate away from its initial plane. After removal of the load, the plate returns to its original undeformed configuration for elastic buckling.

Elastic plate buckling is theoretically defined by the minimum eigenmode and recognized by a sudden loss of stability followed by the rapid development of deflections. After removal of the load, the plate returns to its original undeformed configuration.

The elastic buckling limit can be stable, unstable or neutral according to the load-bearing capacity of the element as shown in Figure 2. A stable limit allows the structure to carry higher loads than the eigenvalue even though large deflection appears. In contrast, an unstable limit does not allow the structure to carry further loads and its capacity drops below the eigenvalue with a fast deflection growth. For a neutral limit, the load-carrying capacity neither increases nor decreases. (DNVGL, 2015).

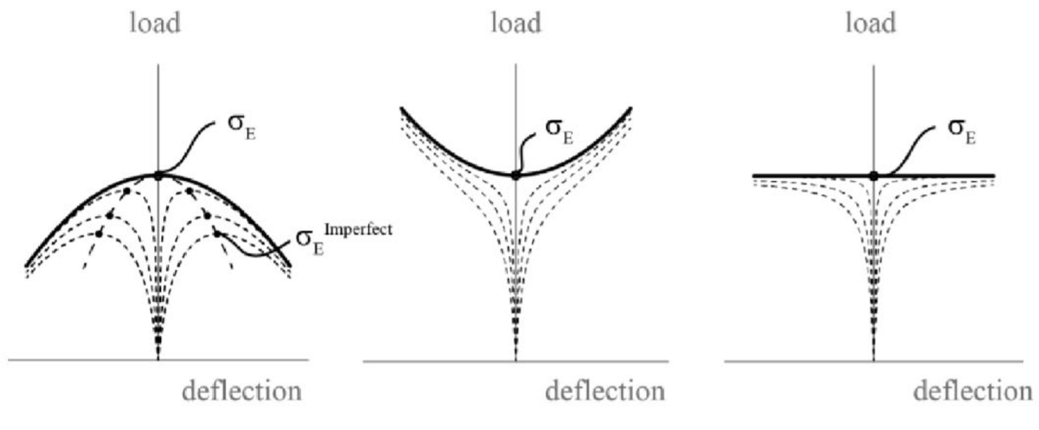


Figure 2. Elastic stability categorization (DNVGL, 2015)

The methods used to analyse elastic plate buckling problems are either by solving the differential equation of equilibrium or applying energy methods and the value of the critical load is given as

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

Where, k is a factor depends on aspect ratio, boundary condition and the load (see Figure 3).

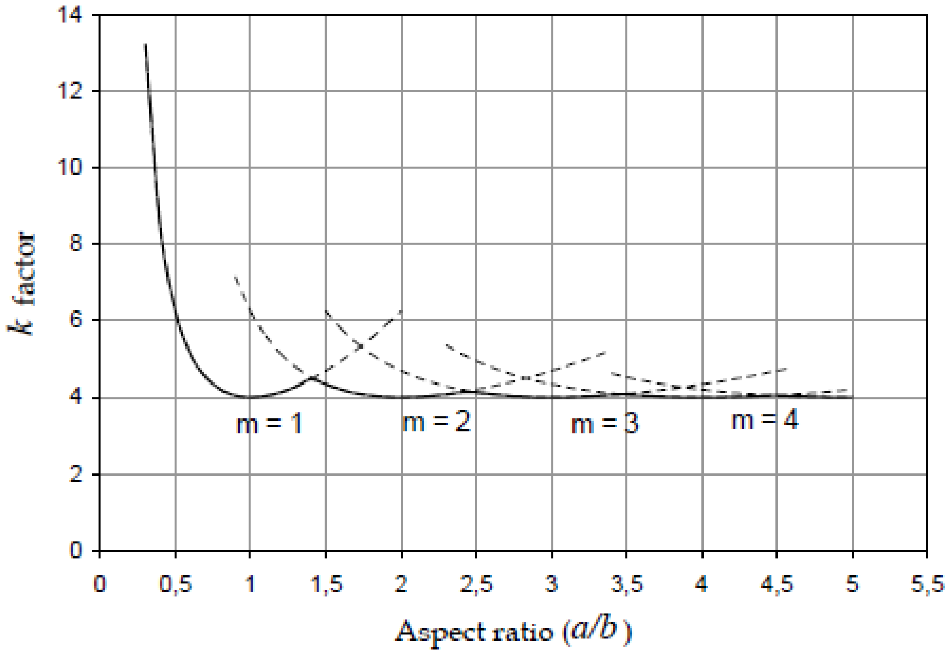


Figure 3. Buckling coefficient versus aspect ratio (Amdahl, 2005)

Correction for Plasticity

The slenderness ratio gives a measure of the failure of a structure being dominated by buckling effects. The slenderness ratio is defined by DNV (DNV, 2010), as

$$\lambda = \sqrt{\frac{\sigma_y}{\sigma_E}} \tag{2}$$

Where,

σ_y = Material yield strength and

σ_E = Euler buckling stress for plates

Based on λ value structures can be classified as follows,

Slender structure $\lambda > 1.4$

Moderate slender structure $0.6 < \lambda < 1.4$

Stocky structures $0.6 > \lambda$

The relation between buckling capacity and slenderness ratio is plotted below, Figure 4

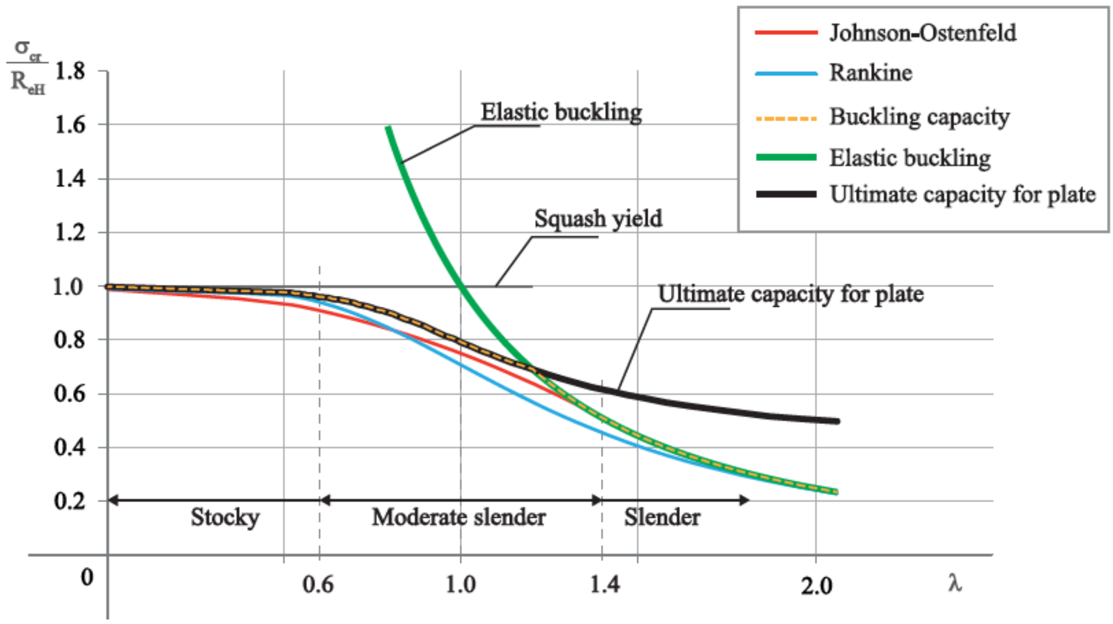


Figure 4. Buckling design curve as a function of the slenderness parameter (DNVGL, 2015)

In the case of plates with low width to thickness ratio, Equation (1) may give a critical stress value greater than yield strength which is not physically feasible. There are different approaches available to consider this plasticity effect in the calculation. A helpful method for adjusting the elastic critical stress due to plasticity is the ϕ method, where the elastic-plastic buckling stress is given by,

$$\sigma_{cr} = \phi\sigma_y \tag{3}$$

where ϕ= empirical function related to slenderness ratio.

The Johnson–Ostenfeld formulation method considers the effects of plasticity in the elastic buckling strength. The resulting "elastic-plastic" buckling strength is termed the "critical" buckling strength, which is approximately regarded as the ultimate strength (Paik, 2018).

The Johnson-Ostenfeld formula provides an expression for ϕ as,

$$\phi = \begin{cases} 1 - \frac{\lambda^2}{4} & for\ \lambda^2 \leq 2 \\ \frac{1}{\lambda^2} & for\ \lambda^2 > 2 \end{cases} \tag{4}$$

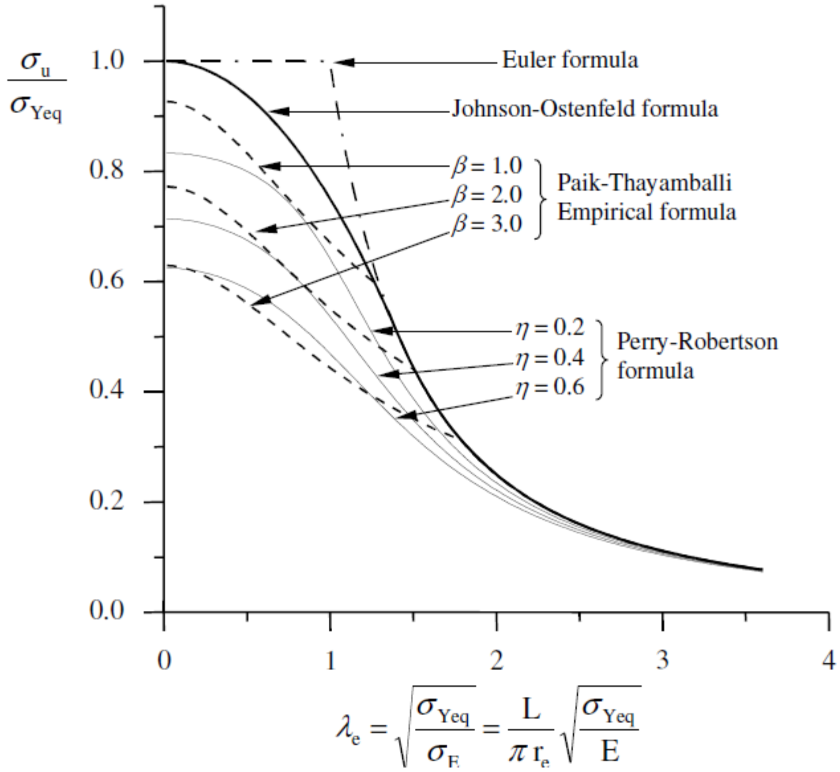


Figure 5. Different methods to correct for plasticity in buckling analysis (Rigo & Rizzutto, 2003)

The critical buckling strength based on the Johnson-Ostenfeld model is therefore calculated from the Equations (3) and (4) to be

$$\sigma_{cr} = \begin{cases} \sigma_E & for\ \ \sigma_E \leq \alpha\sigma_F \\ \sigma_F\left(1 - \frac{\sigma_F}{4\sigma_E}\right) & for\ \ \sigma_E > \alpha\sigma_F \end{cases} \tag{5}$$

Where,

σ_F = Reference yield stress and is σ_y for compressive stress

σ_E = Elastic buckling stress.

2.1.2 Stiffened Plates

The stiffened plate panel is one of the common structural components of marine structures. It contains plates with stiffeners supported by heavier transverse and longitudinal girders. The basic structural configuration of a stiffened panel is shown in Figure 6.

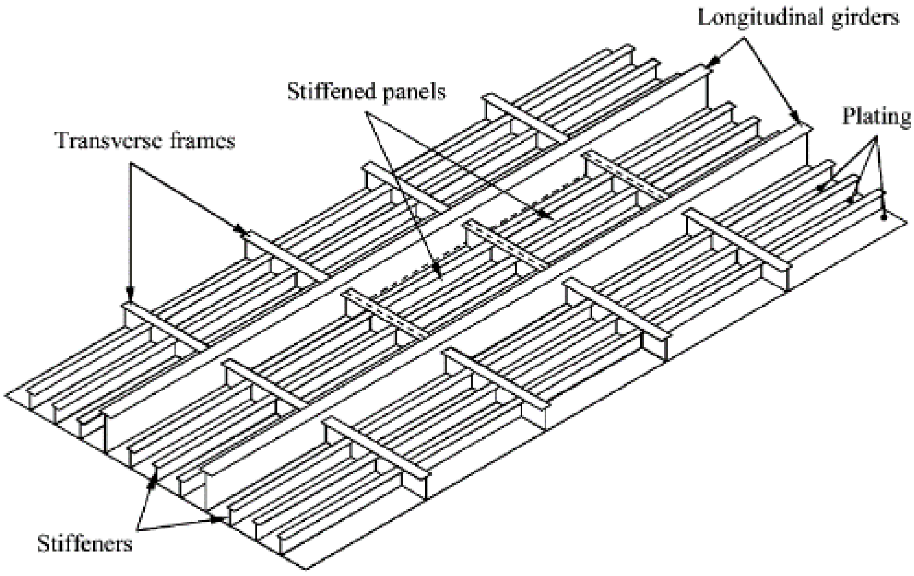


Figure 6. Stiffened Plate Panel (Paik & Kim, 2002)

In longitudinally stiffened deck structures, the stiffeners could be the deck longitudinal and the girders are the heavy beam transverses. In transversely stiffened deck structures, the stiffeners are the deck beams and the transverses are the heavy deck girders.

The primary purpose of the stiffened plate is to transfer the loads acting on the structure to other parts of the vessel. The plates are designed to transfer the hydrostatic loads which arise from the difference between external and internal pressure to the stiffeners, which again, through beam action, transfer the loads to the transverse girders. An example of a stiffened plate under combined load is shown in Figure 7.

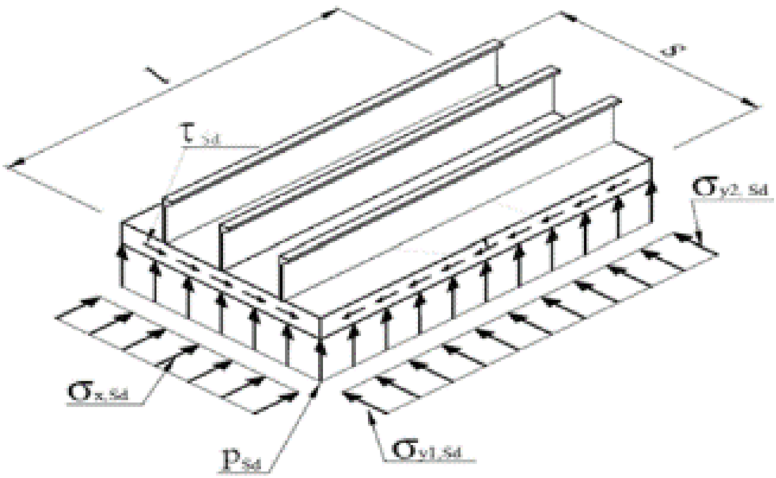


Figure 7. Stiffened plate under combined loads (DNV, 2010)

Figure 7 shows a stiffened plate under combined loads. $\sigma_{x,sd}$ is the axial stress (considered as the uniformly distributed load);$\sigma_{y1,sd}$ and $\sigma_{y2,sd}$ are the stresses in transverse direction (maybe uniformly or linearly distributed loads);τ_{sd} is the shear stress and P_{sd} is the lateral pressure.

In Analysis and Design of Ship Structure (Rigo & Rizzuto, 2003) the ship structure is referred to as a box girder or a hull girder. Stiffened panels constituting the ship hull are often subjected to large in-plane stresses as a result of the longitudinal bending of the hull girder. The most common longitudinal bending behaviour of a ship is as shown in Figure 8. In the first case, deck panels experience compression and bottom panels are in tension. In the latter case wave crest are positioned amidships of the vessel which causes hogging and compression in bottom panels.

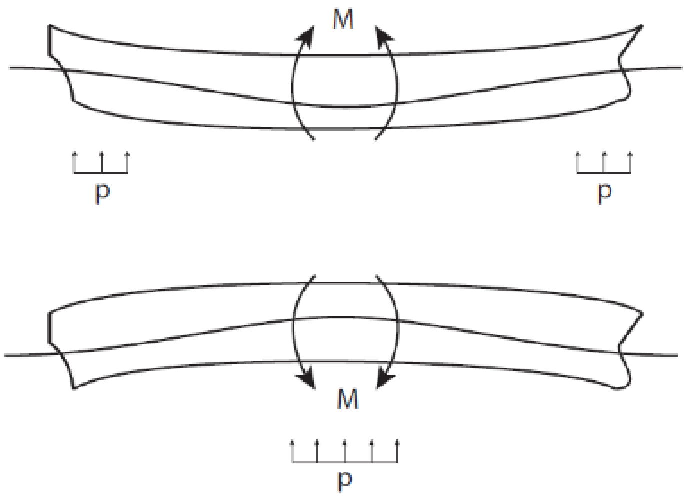


Figure 8. Hull girder bending – In Sagging (above) Hogging (below) (Palm, 2016)

The behaviour of stiffened panels is mainly influenced by several factors which including plate geometry, plate thickness, stiffener geometry, material properties, loading condition, initial imperfections, residual stresses, and existing local damage related to corrosion, fatigue crack and denting.

2.1.3 Mechanism of Plate Buckling

The main assumption used in the estimation of the load-carrying capacity of plating between stiffeners is that the stiffener is stable and fail only after the plating. In general, the ship plating between stiffeners is subjected to mainly four load components, namely longitudinal compression/tension, transverse compression/tension, edge shear and lateral pressure loads (see Figure 9).

To calculate the elastic buckling strength components under single types of loads, that is, σ_{xE} for σ_{xav}, σ_{yE} for σ_{yav} and τ_E for τ_{av}, the related effects arising from in-plane bending, lateral pressure, cut-outs, edge conditions and welding induced residual stresses should be taken into consideration.

The critical (elastic-plastic) buckling strength components under single types of loads, that is, σ_{xB} for σ_{xav}, σ_{yB} for σ_{yav} and τ_B for τ_{av}, are typically calculated by plasticity correction of the corresponding elastic buckling strength using the Johnson-Ostenfeld formula, given by Equation (5).

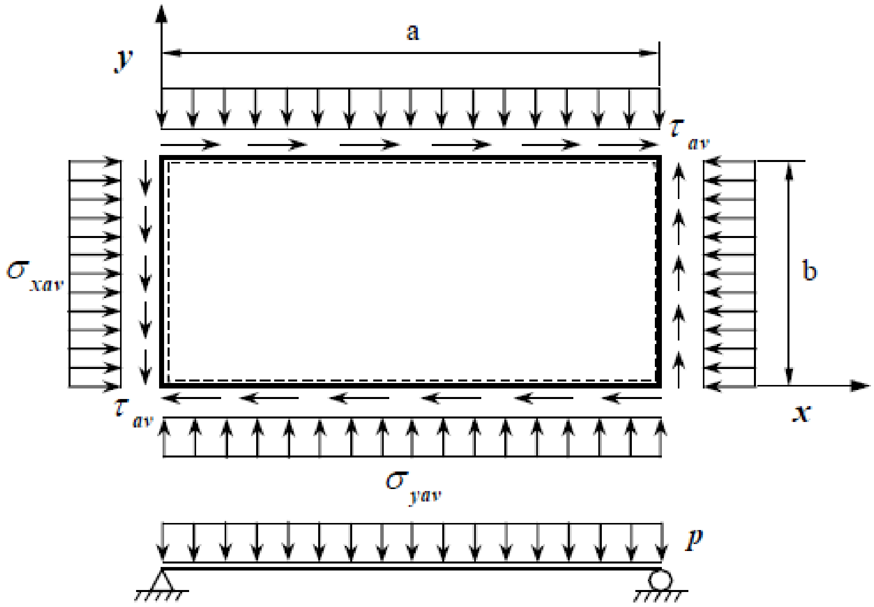


Figure 9. A Simply Supported Rectangular Plate Subject to Biaxial Compression/tension, Edge Shear and Lateral Pressure Loads (Rigo & Rizzutto, 2003)

The critical plate buckling strength must be greater than the corresponding applied stress component with the relevant margin of safety, under single types of loads.

For combined biaxial compression/tension and edge shear, the following critical buckling strength interaction criterion would need to be satisfied.

$$\left(\frac{\sigma_{xav}}{\sigma_{xB}}\right)^c - \alpha \frac{\sigma_{xav}}{\sigma_{xB}} \frac{\sigma_{yav}}{\sigma_{yB}} + \left(\frac{\sigma_{yav}}{\sigma_{yB}}\right)^c + \left(\frac{\tau_{av}}{\tau_B}\right)^c \le \eta_B \qquad (6)$$

Where,

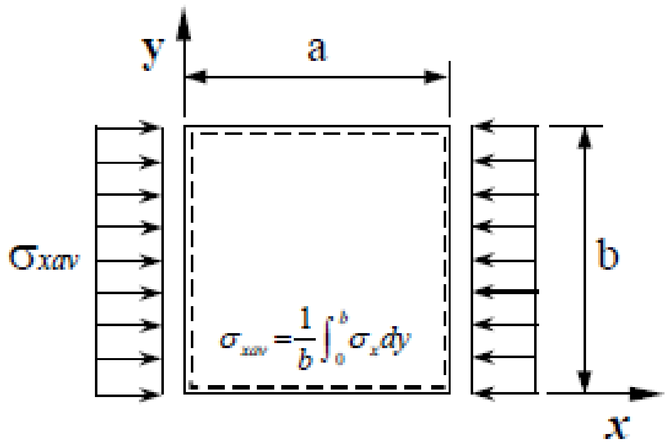
η_B = usage factor for buckling strength, which is typically the inverse of the conventional safety factor.

η_B = 1 is often taken for direct strength calculation, while it is taken less than 1.0 for practical design in accordance with classification society rules. (Rigo & Rizzutto, 2003)

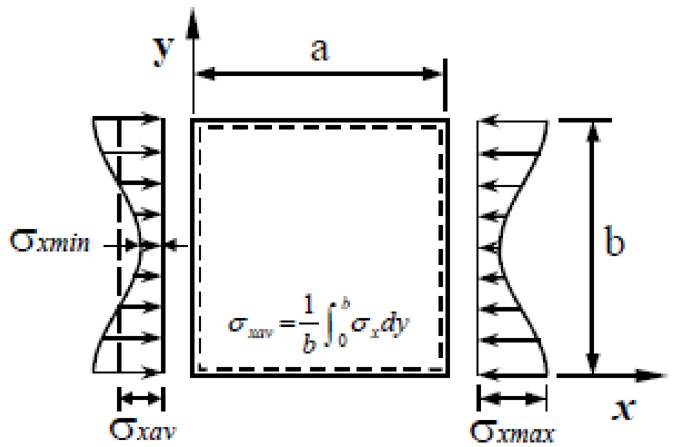
Compressive stress is taken as negative while tensile stress is taken as positive and α=0 if both σ_{xav} and σ_{yav} are compressive, and α=1 if either σ_{xav} or σ_{yav} or both are tensile. The constant c is often taken as c=2.

The membrane stress distribution in the loading direction can become non-uniform as the plate element deforms. Figure 10 indicates a typical example of the axial membrane stress distribution inside a plate element under predominantly longitudinal compressive loading before and after buckling occurs.

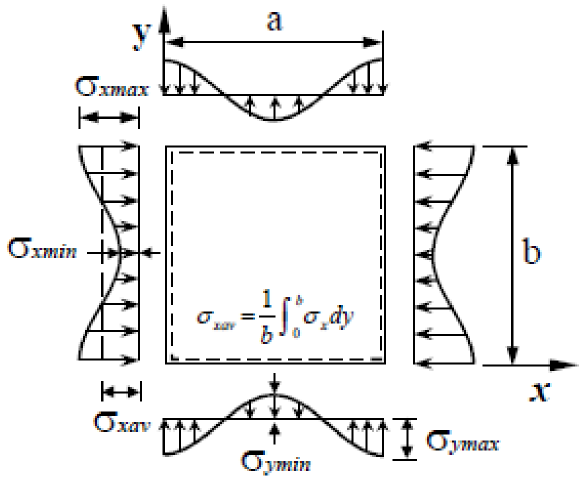
The membrane stress distribution remains uniform before the buckling starts. When buckling starts the stress distribution becomes non-uniform in the loading direction. If the edges of the unloaded plate are free to move in-plane, no membrane stresses will develop in the y-direction and become non-uniform if the unloaded plated edges remain straight as shown in Figure 10.



a) Before buckling



b) After buckling, unloaded edges move freely in plane



c) After buckling, unloaded edges kept straight

Figure 10. Membrane stress distribution inside the plate element under predominantly longitudinal compressive loads (Rigo & Rizzutto, 2003)

With an increase in the deflection of the plate keeping the edges straight, the upper and/or lower fibres inside the middle of the plate element will initially yield by the action of bending. However, as long as it is possible to redistribute the applied loads to the straight plate boundaries by the membrane action, the plate element will not collapse. The collapse will then occur when the most stressed boundary locations yield since the plate element cannot keep the boundaries straight any further, resulting in a rapid increase of lateral plate deflection.

2.1.4 Buckling Failure Modes

According to (Paik & Kim, 2002), the primary modes for the failure with regards to the structure's ultimate limit state of a stiffened panel subject to predominantly axial compressive loads may be categorized as follows:

- Mode I: Overall collapse after overall buckling of the plating and stiffeners as a unit., Figure 11(a)
- Mode II: Plate-induced failure by yielding at the corners of plating between stiffeners, Figure 11(b)
- Mode III: Plate-induced failure by yielding of plate–stiffener combination at midspan, Figure 11(c)
- Mode IV: Stiffener induced failure by local buckling of the stiffener web, Figure 11(d)

- Mode V: Stiffener-induced failure by lateral-torsional buckling of stiffener, Figure 11(e) and
- Mode VI: Gross yielding.

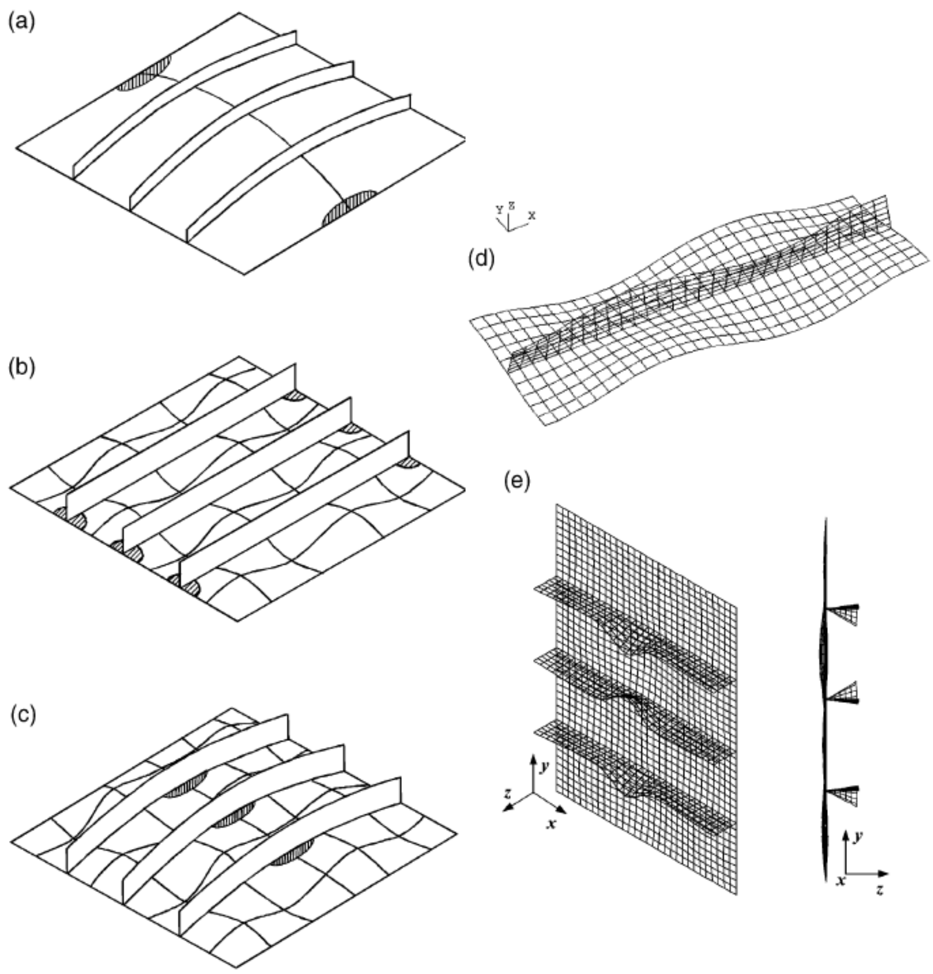


Figure 11. Failure modes of stiffened plate subjected to loading (Paik & Kim, 2002)

From Figure 11 (a) Mode I: Overall collapse after overall buckling of the plating and stiffeners as a unit. (b). Mode II: Plate-induced failure by yielding at the corners of plating between stiffeners. (c). Mode III: Plate-induced failure by yielding of plate-stiffener combination at mid-span. (d). Mode IV: Stiffener induced failure by local buckling of the stiffener web. (e). Mode V: Stiffener-induced failure by lateral- torsional buckling of stiffener.

As stated by Paik (Paik & Kim, 2002), In Mode I, the stiffeners may buckle together with the plating when the stiffeners are relatively weak and the overall buckling behaviour remains elastic. The panel behaves as an 'orthotropic plate' in this mode. The stiffened panel can normally sustain further loading even after overall buckling in the elastic regime occurs and the ultimate strength is eventually reached by the formation of a large yield region inside the panel and/or along the panel edges.

Mode II typically represents the collapse pattern wherein the panel collapses by yielding at the corners of plating between stiffeners, which is usually termed a plate induced failure at ends. This type of collapse can also occur in some cases when the panel is predominantly subjected to biaxial compressive loads.

Mode III indicates a failure pattern in which the ultimate strength is reached by yielding of the plate–stiffener combination at the mid-span. Mode III failure typically occurs when the dimensions of the stiffeners are intermediate, that is, neither weak nor very strong.

Modes II-VI normally takes place when the stiffeners are relatively strong so that the stiffeners remain straight until the plating between stiffeners buckles or even collapses locally.

Modes IV and V failures typically arise when the ratio of stiffener web height to stiffener web thickness is too large and/or when the type of the stiffener flange is inadequate to remain straight so that the stiffener web buckles or twists sideways. Mode IV represents a failure pattern in which the panel collapses by local buckling of stiffener web, while Mode V can occur when the ultimate strength is reached by lateral–torsional buckling (also called tripping) of stiffener.

Mode VI typically takes place when the panel slenderness is very small (i.e., the panel is very stocky or thick) and/or when the panel is predominantly subjected to the axial tensile loading so that neither local nor overall buckling occurs until the panel cross section yields entirely.

2.2 Redundancy of Structures and Progressive Collapse Analysis

Redundancy of a component or a system is defined as the capability to keep up or reestablish its function when a failure of a member or connection has occurred. It can be accomplished by strengthening the part/component or introducing an alternate load path.

It has become a challenge for structural and design engineers to design safe and robust marine structures for the uncertain behaviours of such structures. Redundancy or complete damage of structural members are salient issues and a ship and offshore structure could be disposed of due to collapse(Azad et al., 2019).

Redundancy can be defined as the creating of an alternative load path due to some local damages in the structure. It is important to check whether the acting loads are being transferred to other adjacent parts if any local member is fully damaged or partially damaged (Azad et al., 2019).

As stated by the Ship Structure Committee (Ship Structure Committee, 1992), structure redundancy can be classified into the following categories,

- Local Redundancy

Local redundancy refers to local reserve strength which exists in individual structural members or joints. This is the margin between the demand imposed by the loads and the capacity of the structural members or joints.

- Global Redundancy

Global redundancy refers to the overall structure. It is expressed in two forms, system reserve strength and residual strength.

Reserve strength is the margin between the design load and the ultimate capacity of the overall structure to sustain the applied loads. The residual strength is that strength remaining in the structure after one or more components have failed or become ineffective due to damage.

Based on the structural behaviour, a structure can be mainly classified as a statically determinate or a statically indeterminate structure. In a statically determinate structure, when any component reaches its failure load, overall structural collapse or failure occurs. But in a statically indeterminate structure, internal load re-distribution takes place following the local failure of an individual component (Ship Structure Committee, 1992).

Ship structures are assumed as multiple redundant, which leads to the assumption that in the case of buckling no total collapse will occur. The main objective of this thesis is to analyse the redundancy of the structural system in case of panel failure and resulting in progressive collapse path.

2.2.1 Utilization Factor

The utilization factor, also referred to as usage factor, is defined as the ratio between the applied loads and the corresponding buckling limit. It follows that the most general definition is

$$\eta = \frac{Load}{Ultimate\ Capacity}$$

where η is the usage factor for buckling strength. It is the inverse of the safety factor – as higher the safety factor of a structure lowers the utilization factor and vice versa. A highly utilized panel experiences higher stress values and increasing the load on a highly stressed panel can result in structural failure. In this thesis, the following expression from (DNV, 2010) is used to evaluate the utilization factor of the structure.

$$\left(\frac{\sigma_{X,Sd}}{\sigma_{X,Rd}}\right)^2 + \left(\frac{\sigma_{y,Sd}}{\sigma_{y,Rd}}\right)^2 - C_i \left(\frac{\sigma_{X,Sd}}{\sigma_{X,Rd}}\right)\left(\frac{\sigma_{y,Sd}}{\sigma_{y,Rd}}\right) + \left(\frac{\tau_{Sd}}{\tau_{Rd}}\right)^2 \leq 1.0 \qquad (7)$$

According to conventional design norms, the value of the expressions on the left of the expression are not to exceed 1, however, in this thesis, this criteria value will be reduced in order to study the progressive failure of the panels.

2.2.2 Progressive Collapse Analysis

Structural Limit States

A limit state is defined as a condition for which a particular structural member or an entire structure fails to perform the function that it has been designed for beforehand (Paik & Kim, 2002). The design condition may refer to a degree of loading or other actions on the structure, while the criteria refers to the structural integrity, fitness for use, durability, or other design requirements.

According to Rigo & Rizzutto (Rigo & Rizzutto, 2003), classically, the different limit states were divided into 2 major categories: the service limit state and the ultimate limit state. Today, from the viewpoint of structural design, it seems more relevant to use for the steel structures four types of limit states, namely:

1) Service or serviceability limit state

A service or serviceability limit state corresponds to the situation where the structure can no longer provide the service for which it was conceived - the limit state so as not lead to the collapse of the vessel.

2) Ultimate limit state,

An ultimate limit state represents the collapse of the structure due to a loss of structural stiffness and strength. A classic example of the ultimate limit state is the ultimate hull bending moment (Figure 12. The Moment Curvature curve(M-ϕ)(Rigo & Rizzutto, 2003). The point C, which is the highest point of the curve represents the ultimate limit state.

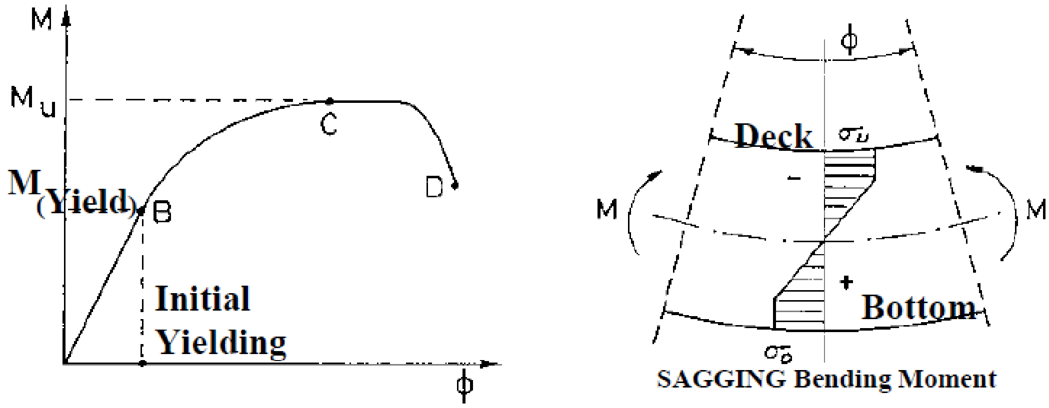


Figure 12. The Moment Curvature curve(M-ϕ)(Rigo & Rizzutto, 2003)

Figure 12 represents the moment-curvature plot. It shows that in sagging, the deck is in compression and reaches the ultimate limit state when $\sigma_{deck} = \sigma_u$ and the bottom is in tension and reached its ultimate limit state after complete yielding, i.e., $\sigma_{bottom} = \sigma_0$ where σ_0 indicates the yield stress.

There are two primary ways to evaluate the hull girder ultimate strength of a ship's hull under longitudinal bending moments, namely the approximate analysis and the progressive collapse analysis. In the approximate analysis, the ultimate bending moment is calculated directly (Mu, Point C on Figure 12). On the other hand, in the second method, progressive collapse analysis is to be performed on a hull girder to obtain both Mu and the curves M-φ (Rigo & Rizzutto, 2003).

3) Fatigue limit state

Fatigue can be considered as a service limit state, as even if it is a matter of discussion, yielding should be considered as a service limit state. The first yield is sometimes used to assess the ultimate state, for instance for the ultimate hull bending moment but basically, collapse occurs later. Most of the time vibration relates to service limit states.

4) Accidental limit state

Collision and grounding some of the conditions which form the accidental limit state. The primary concern of the accidental limit state design in such cases is to maintain the water-tightness of ship compartments, the containment of dangerous or pollutant cargoes, and the integrity of critical spaces at the greatest possible levels, and to minimize the release/outflow of cargo.

For different limit states, various safety levels may be considered for evaluation. In practice, it is important to differentiate service, ultimate, fatigue and accidental limit states because the partial safety factors associated with these limit states are generally different. (Rigo & Rizzutto, 2003) .In this thesis – the ultimate limit state is used along with the progressive collapse analysis.

Progressive collapse is caused by a series of structural element failures due to large internal loads that exceed the elements' bearing capacities. The local damage that initiates the progressive collapse is called initiating damage (Marjanishvili, 2004). Progressive collapse occurs when relatively local structural damage causes a chain reaction of structural element failures, disproportionate to the initial damage, resulting in a partial or full collapse of the structure.

2.2.3 Methods to Evaluate Progressive Collapse Analysis

In ship structural design, the prediction of the progressive collapse behaviour of the ship's hull subjected to longitudinal bending is a significant aspect. This has resulted in extensive research works performed on buckling/plastic collapse of isolated plates, stiffened plate assemblages and overall progressive collapse behaviour of box girders as well as ship's hulls. Existing methods for progressive collapse analysis are typical numerical analysis such as the Finite Element Method (FEM), the Idealized structural Element method (ISUM) and Smith's method.

Finite Element Method (FEM)

The nonlinear finite element method (NLFEM) is a powerful technique used to simulate the nonlinear structural response. It is the most rational way to evaluate the ultimate hull girder strength through a progressive collapse analysis on a ship's hull girder. Both material and geometrical nonlinearities can be considered.

It requires enormous modelling effort and computing time for non-linear analysis of large structures. For this reason, the analysis is more conveniently performed on a section of the hull that sufficiently extends enough in the longitudinal direction to the model the characteristic behaviour. The typical analysis may concern one frame spacing of a whole compartment. These analyses have to be supplemented by information on the bending and shear loads that act at the fore and aft transverse loaded sections. Finite Element Analysis (FEA) has shown that accuracy is limited because of the boundary conditions along the transverse sections where the loading is applied, the position of the neutral axis along the length of the analysed section and the difficulty to model the residual stresses (Rigo & Rizzutto, 2003), along with additional loss of accuracy due to model simplification for computational purposes.

Idealized Structural Unit Method (ISUM)

The idealized structural unit method (ISUM) was developed by Ueda and Rashed (Ueda & Rashed, 1984) as a simplified approach with reduced computational efforts compared to non-linear finite element method. Their first effort on ISUM was to perform the progressive collapse analysis of a ship transverse framed structure as an assembly of the "deep girder" ISUM elements.

The non-linear behaviour of each type of structural member, namely, beams, columns, rectangular plates and stiffened panels is idealized and expressed in the form of a set of failure functions defining the necessary conditions for different failures which may take place in the corresponding ISUM unit, and sets of stiffness matrices representing the nonlinear relationship between the nodal force vector and the nodal displacement vector until the limit state are reached (Rigo & Rizzutto, 2003).

Smith's Method

This method is a simplified procedure to perform progressive collapse analysis, and it offers computational efficiency and sufficient accuracy of the predicted structural responses. This is a simple and efficient method for progressive collapse analysis of box girders including ship's hulls subjected to pure longitudinal bending. It involves mesh modelling the structure with smaller elements, deriving an 'Average stress-strain relationship' between each element and thereby carry out the progressive analysis. (Smith, 1966) (Figure 13):

The characteristic of each step has been described as detail in (Rigo & Rizzutto, 2003).

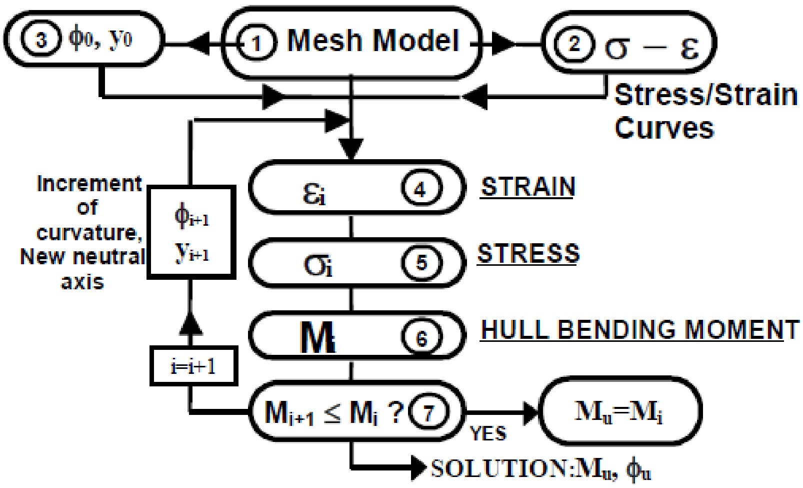


Figure 13. Smith's Progressive Collapse Method (Rigo & Rizzutto, 2003)

2.2.4 Numerical Modelling Approach Used

In this thesis, the Finite Element method is used to evaluate stress values and thereby the progressive collapse of a statically indeterminate structure. The structure is considered to be made up of several panels between the longitudinal and transverse girders (see Figure 14). The buckling resistance and usage factor parameters at each panel is evaluated by the DNV-RP-C201 expressions, which are detailed in the next chapter. The stress values on these panels obtained are used to calculate the structure's usage factor and based on this factor – the integrity of the structure is investigated.

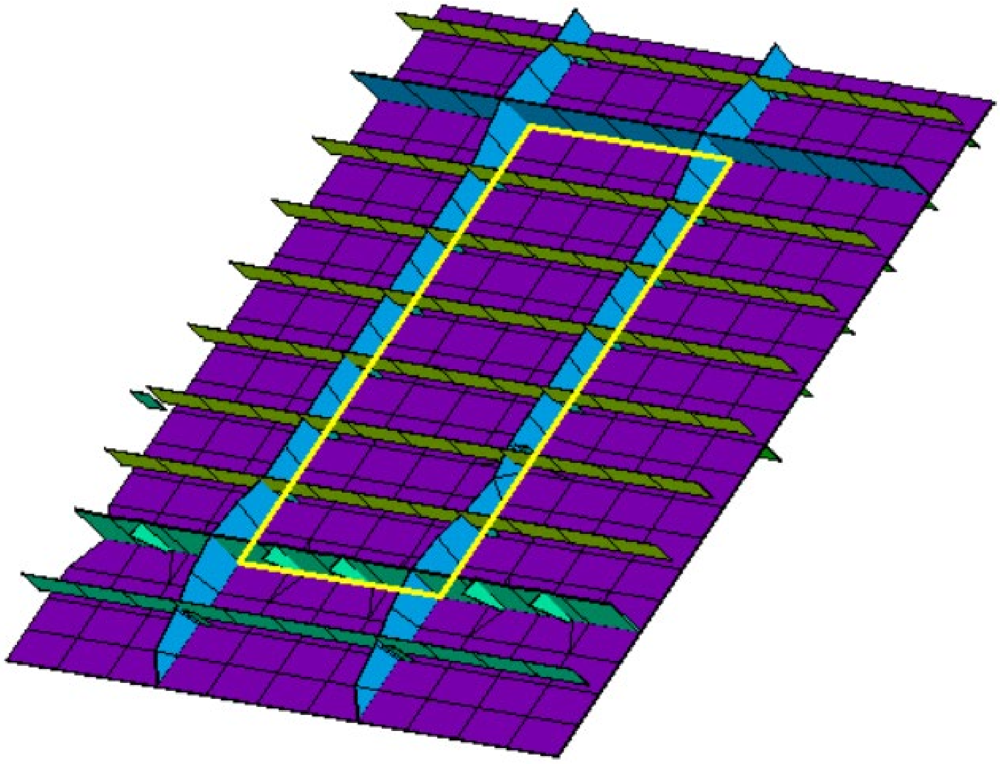


Figure 14. A Stiffened Panel (Highlighted)

In order to develop a working model of the code a simplistic sample model was used, and with the developed code – several analyses were carried out on a section of a Cruise Vessel. Details of the development of this code and the result of this analysis are presented in succeeding sections.

2.2.5 Parameter Sensitivity Analysis of the Buckling Resistance

As it has been detailed, the utilization factor and the respective buckling resistances have been considered an important parameter to evaluate the structural integrity of the panel. Therefore, an in-depth analysis into the different parameters that influence the buckling resistances has been carried out. In order to carry out this parameter sensitivity, the different buckling resistances expressions detailed earlier are used, and their sensitivity to parameters such as plate thickness, panel width, stiffener spacing are presented for the following loading conditions:

Unstiffened Panel	Stiffened Panel
Longitudinal Compression	Longitudinal Compression
Transverse Compression	Transverse Compression
Shear Compression	-

The procedure and results of this analysis are presented in succeeding sections.

3 RULE-BASED BUCKLING STRENGTH ANALYSIS

The DNV-GL expressions to evaluate the buckling of plates under different compression loads is taken from *DNV-RP- C201- Buckling Strength of Plated Structures*(DNV, 2010) and are the expressions used to evaluate the different structures analysed in this thesis. The recommendation for buckling of unstiffened plates is based on established design methods using the post-critical capacity of plates often presented as the effective width method and buckling of stiffened panels are treated by transforming the buckling problem of the panel to a buckling problem of a beam-column (strut model) (DNV, 2010).

Effective Width Concept

For the estimation of plate ultimate strength components under uni-axial compressive loads, different approaches are available. The Effective Width Method is one of the widely used methods used to evaluate plate buckling, which is by assuming the plate will collapse when the maximum compressive stress at the corner of the plate reaches the yield stress of the material.

In the case of slender plates, the maximum load the plates can withstand is significantly higher than the value expected by elastic theory provided that their edges are constrained to remain straight. Membrane stresses that develop in the transverse direction as a result of large lateral deflections, tends to stabilize the plates.

According to the effective width concept, the plate ultimate strength components under uniaxial compressive loads (σ_{xu} *and* σ_{yu}) are given as:

$$\frac{\sigma_{xu}}{\sigma_Y} = \frac{b_{eu}}{b} \quad \text{and} \quad \frac{\sigma_{yu}}{\sigma_Y} = \frac{a_{eu}}{a} \qquad (8)$$

Where,

a_{eu}= plate effective length at ultimate limit state

b_{eu}= plate effective width at ultimate limit state

Faulkner (Faulkner ,1975) suggested an empirical formula for effective width $\left(\frac{b_{eu}}{b}\right)$ for simply supported steel plates given by,

- for Longitudinal Axial Compression

$$\frac{b_{eu}}{b} = \begin{cases} 1\, for\, \beta < 1 \\ \frac{C_1}{\beta} - \frac{C_2}{\beta^2}\, for\, \beta \geq 1 \end{cases} \tag{9}$$

- for Transverse Axial Compression

$$\frac{a_{eu}}{a} = \frac{0.9}{\beta^2} + \frac{b}{a}\frac{1.9}{\beta}\left(1 - \frac{0.9}{\beta^2}\right) \tag{10}$$

Where,

$\beta =$ plate slenderness, given by

$$\beta = \frac{b}{t}\sqrt{\frac{\sigma_Y}{E}} \tag{11}$$

E = Young's modulus

t = Plate Thickness

C_1 =2 and C_2=1 for plates simply supported at all (four) edges or C_1 =2.25 and C_2=1.25 for plates clamped at all edges.

By substituting the plate effective width formula into Equation (1), the plate ultimate strength components can be estimated.

In the post-stability range, it is necessary to distinguish between compression stress in the longitudinal and transverse direction of a rectangular plate field as well as shear stress. For this reason, different capacity formulations (buckling curves) are given for the different stress component.

The reduction in plate resistance for in-plane compressive forces is expressed by a reduced (effective) width of the plate which is multiplied by the design yield strength to obtain the design resistance.

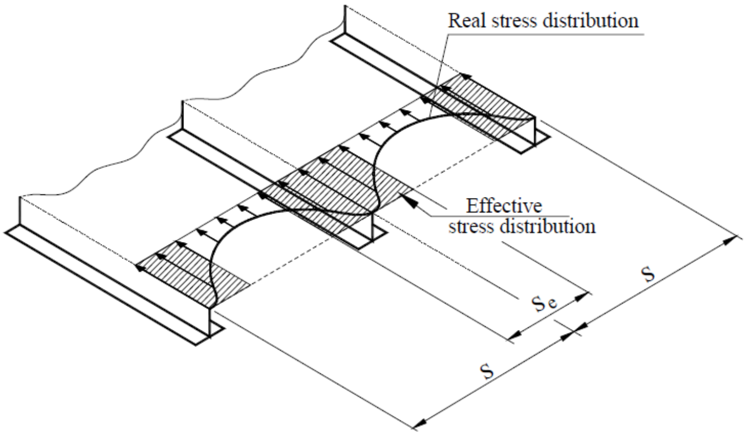


Figure 15. Concept of Effective width method (DNV, 2010)

Ideal Elastic-Plastic Strut Analysis

Buckling of stiffened panels are treated by the DNV RP by transforming the buckling problem of the panel to a buckling problem of a beam-column (strut model) as illustrated in Figure 16. A stiffener with its associated plate flange is conveniently modelled as an equivalent beam-column.

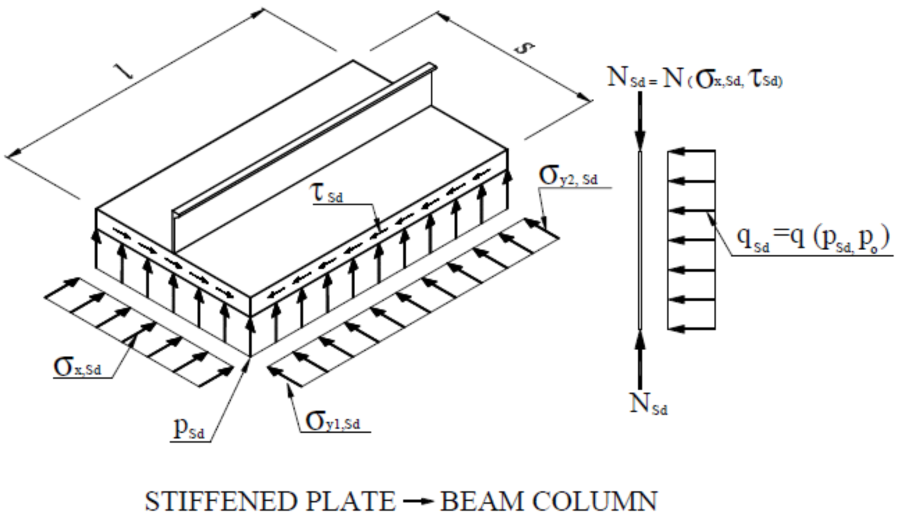


Figure 16. Strut Model (DNV, 2010)

The longitudinal compression and shear affecting the stiffened panel are considered by transferring and combining them to an equivalent axial force. This axial force is represented by in Figure 16 The lateral pressure p_0 and the design lateral pressure p_{sd} are taken into account by combining them to an equivalent lateral line load.

Because of the nature of applied axial compressive loading, the possible yield locations are longitudinal mid-edges for longitudinal uni-axial compressive loads and transverse mid-edges for transverse uni-axial compressive loads (Solland & Jensen, 2004).

The RP-C201 document gives design recommendations to flat steel plate structures intended for marine structures. The plate panel maybe the web or the flange of a beam, or a part of box girders, bulkheads, pontoons, hull or integrated plated decks (Nguyen, 2011). The main assumption used in the design recommendations in the Recommended Practise (RP) is a hierarchy of structural components where the lower component is assumed to be supported by the next member in the following hierarchy as

1. Plate between stiffener
2. Stiffener
3. Girder

The buckling resistance expressions do not account for material non-linearities, imperfections, residual stresses and possible interaction between the global and local buckling.

a) Buckling of Plates Under Longitudinally Uniform Compression

The design buckling resistance under longitudinal compression force is calculated as follows,

$$\sigma_{X,Rd} = C_x \frac{f_y}{\gamma_M} \tag{12}$$

Where

$$\begin{cases} C_x = 1 & when\ \lambda_p \leq 0.673 \\ C_x = \dfrac{(\lambda_p - 0.22)}{\lambda_p^2} & when\ \lambda_p > 0.673 \end{cases} \tag{13}$$

with plate slenderness λ_p, given by,

$$\lambda_p = \sqrt{\frac{f_y}{f_{cr}}} = 0.525 \frac{s}{t} \sqrt{\frac{f_y}{E}} \tag{14}$$

In which

s = Stiffener Spacing

t = Plate Thickness

f_{cr} = Critical Plate Buckling Strength

The resistance of the plate is satisfactory when:

$$\sigma_{x,Sd} \leq \sigma_{x,Rd} \tag{15}$$

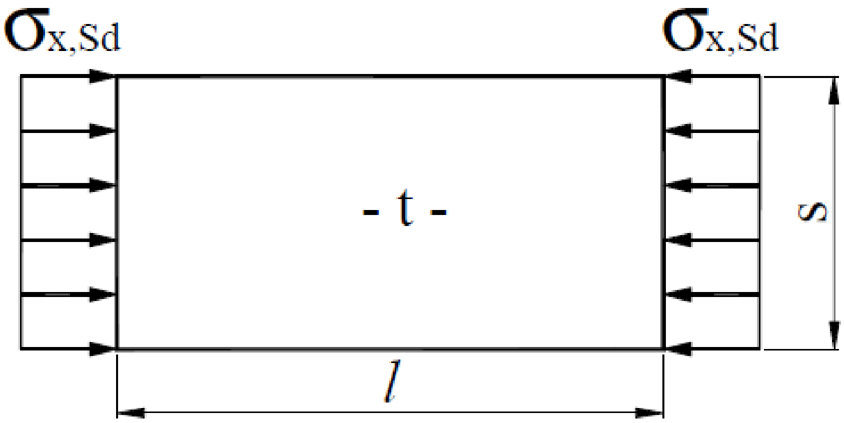


Figure 17. Plate with Longitudinal Compression (DNV, 2010)

b) Buckling of Plates with Transverse Compression

The design buckling resistance under transverse compression force may be found from:

$$\sigma_{y,Rd} = \frac{\sigma_{y,R}}{\gamma_M} \tag{16}$$

$$\sigma_{y,R} = \left[\frac{1.3\,t}{l} \sqrt{\frac{E}{f_y}} + \kappa \left(1 - \frac{1.3\,t}{l} \sqrt{\frac{E}{f_y}} \right) \right] f_y\, k_p \tag{17}$$

where,

$$\begin{cases} \kappa = 1 & for\ \lambda_c \leq 0.2 \\ \kappa = \dfrac{1}{2\,\lambda_c^2} \left(1 + \mu + \lambda_c^2 - \sqrt{\left(1 + \mu + \lambda_c^2\right)^2 - 4\,\lambda_c^2} \right) & for\ 0.2 < \lambda_c < 2 \\ \kappa = \dfrac{1}{2\,\lambda_c^2} + 0.07 & for\ \lambda_c \geq 2 \end{cases} \tag{18}$$

where, λ_c is:

$$\lambda_c = 1.1\, \frac{s}{t} \sqrt{\frac{f_y}{E}} \tag{19}$$

And μ is:

$$\mu = 0.21\ (\lambda_c - 0.2) \tag{20}$$

where,

t = plate thickness

l = plate length

s = plate width

The resistance of the plate is satisfactory when:

$$\sigma_{y,Sd} \leq \sigma_{y,Rd} \tag{21}$$

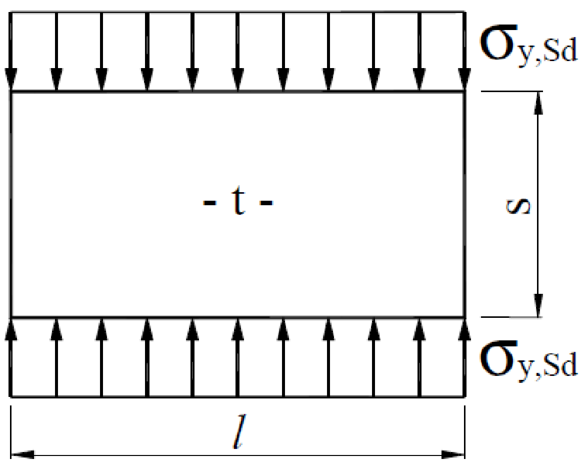


Figure 18. Plate with Transverse Compression (DNVGL, 2015)

c) Buckling of Plate with Shear

Shear buckling of a plate can be checked by

$$\tau_{Sd} \leq \tau_{Rd} \tag{22}$$

Where, τ_{Rd} is design resistance shear stress in cases $\sigma_{y,Sd}$ is zero or negative (tension) which is calculated by,

$$\tau_{Rd} = \frac{C_\tau}{\gamma_M} \frac{f_y}{\sqrt{3}} \tag{23}$$

With

$$\begin{cases} C_\tau = 1 & for\ \lambda_w \leq 0.8 \\ C_\tau = 1 - 0.625(\lambda_w - 0.8) & for\ 0.8 < \lambda_w \leq 1.2 \\ C_\tau = \dfrac{0.9}{\lambda_w} & for\ \lambda_w > 1.2 \end{cases} \tag{24}$$

And reduced slenderness, λ_w is obtained from:

$$\lambda_w = 0.975\ \frac{s}{t}\sqrt{\frac{f_y}{E\ k_l}} \tag{25}$$

Where,

$$\begin{cases} k_l = 5.34 + 4\left(\dfrac{s}{l}\right)^2 & for\ l \geq s \\ k_l = 5.34\left(\dfrac{s}{l}\right)^2 + 4 & for\ l < s \end{cases} \tag{26}$$

In case $\sigma_{y,Sd}$ is positive (compression), then τ_{Rd} is calculated by

$$\tau_{Rd} = \frac{C_{\tau e}}{\gamma_M}\frac{f_y}{\sqrt{3}} \tag{27}$$

With

$$\begin{cases} C_{\tau e} = 1 & for\ \lambda_w \leq 0.8 \\ C_{\tau e} = 1 - 0.8(\lambda_w - 0.8) & for\ 0.8 < \lambda_w \leq 1.25 \\ C_{\tau e} = \dfrac{1}{\lambda_w^2} & for\ \lambda_w > 1.25 \end{cases} \tag{28}$$

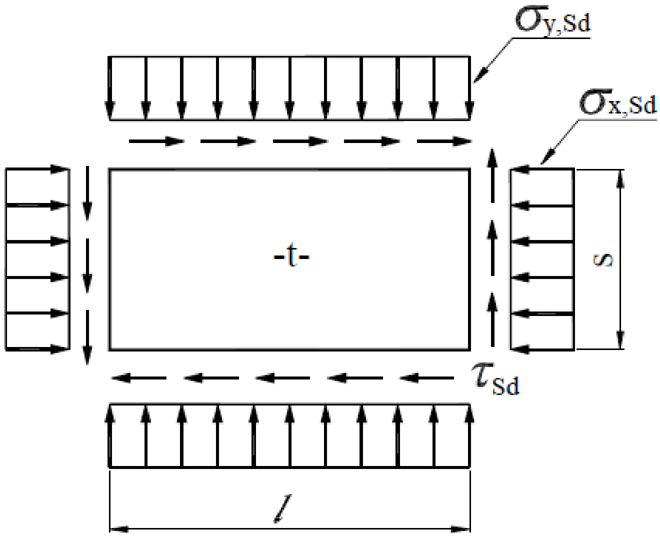


Figure 19. Bi-axially loaded plate with shear (DNV, 2010)

d) Buckling of Biaxially Loaded Plates with Shear – Usage Factor

A plate subjected to biaxially loading with shear should fulfil the following requirement:

$$\left(\frac{\sigma_{X,Sd}}{\sigma_{X,Rd}}\right)^2 + \left(\frac{\sigma_{y,Sd}}{\sigma_{y,Rd}}\right)^2 - C_i \left(\frac{\sigma_{X,Sd}}{\sigma_{X,Rd}}\right)\left(\frac{\sigma_{y,Sd}}{\sigma_{y,Rd}}\right) + \left(\frac{\tau_{Sd}}{\tau_{Rd}}\right)^2 \leq 1.0 \qquad (29)$$

Where,

If both $\sigma_{X,Rd}$ and $\sigma_{y,Rd}$ is compressive (positive), then

$$\begin{cases} C_i = 1 - \frac{s}{120\,t} & for\ \frac{s}{t} \leq 120 \\ C_i = 0 & for\ \frac{s}{t} > 120 \end{cases} \qquad (30)$$

If either of $\sigma_{X,Rd}$ or $\sigma_{y,Rd}$ is in tension(negative), then C_i=1.0

The left-hand side of the Equation (29) is a crucial parameter called usage factor and used widely in this thesis for structural buckling analysis. Details regarding the Utilization Factor will be elaborated in succeeded sections.

4 METHODOLOGY FOR NUMERICAL MODELLING TO EVALUATE PROGRESSIVE STRUCTURAL COLLAPSE

4.1 Numerical Modelling to Evaluate Progressive Structural Collapse

Numerical modelling is used to evaluate the various problem statements studied in this thesis. ANSYS is used, along with user-defined macros to load, analyse and extract the data from the program. When a ship hull is under vertical bending, the decks are predominantly subjected to longitudinal axial compression in sagging or longitudinal axial tension in hogging, while bottom panels are subjected to combined longitudinal axial compression/tension and lateral pressure. Upper side shells are normally subjected to combined longitudinal axial compression/tension and longitudinal in-plane bending, while lower side shells are subjected to combined axial compression/tension, longitudinal in-plane bending and lateral pressure loads (Paik & Kim, 2002). Numerical modelling can provide reliable solutions for such a complex problem evaluation.

Numerical simulations have been carried out to evaluate the usage factor of different structures under different loading conditions, and thereby the progressive collapse of the structure. As the global loads applied to the structure cannot be varied, the cut-off utilization factor value is reduced in order to study the progressive collapse path.

In this thesis, all the numerical simulations have been performed with ANSYS APDL. The procedures followed are explained in the following sub-chapters, starting with the general layout of ANSYS Mechanical APDL version 18.2, Research version, that has been used to fulfil this thesis.

4.1.1 ANSYS Parametric Design Language

APDL is an acronym for the ANSYS Parametric Design Language (APDL) and it is a powerful scripting language that allows to parameterize the model and automate common tasks. It is one of the most powerful features of ANSYS. It allows the user to define some or all parts of the model (geometry, material properties, loads, etc.) as parameters. Creating and solving a new variation of a parameterized model is as simple as changing a few parameter values and rerunning the model. This makes ANSYS a powerful tool for engineering analysis, optimization, root cause analysis, and for the design of new systems and technologies. APDL also allows the user to build and execute macros, run macros as ANSYS commands, operate on parameter arrays, and do simple logic (if, then, else, do, repeat, etc.)(Thompson & Thompson, 2017).

The main features of APDL are (UNICAMP, n.d.),

- Input model dimensions, material properties, etc in terms of parameters rather than numbers.
- Retrieve information from the ANSYS database, such as node location or maximum stress.
- Perform mathematical calculations among parameters, including vector and matrix operations.
- Define abbreviations (short cuts) for frequently used commands or macros.
- Create a macro to execute a sequence of tasks, with if then else branching, do loops, and user prompts.

The main features of APDL used in this thesis work were the creation of user-defined function which was used to implement the DNVGL- buckling strength analysis equations, retrieving of information from ANSYS databases such as stress values, thickness, area, the moment of inertia and creation of macros to execute a sequence of tasks, and interaction with GUI (Graphical user interface) options.

4.1.2 Progressive Collapse Analysis

Redundancy analysis is performed for the vessel structure through several simulations and the methodology of the performing the simulations are presented here. Using the various macros delineated in the previous section – several simulations are run to evaluate the progressive critical path of failure under different loadings.

Possible Case Scenarios

The following assumptions were considered while performing the Progressive Collapse analysis:

- The utilization factor value is calculated and if the value is greater than the selected criteria, the structure is assumed to have no load-carrying capacity.

- When a panel is removed at the end of each iteration, a part of the load also gets removed. The total load applied varies slightly over each iteration as the panels removed, however, this does not affect the final result values significantly.

After each iteration panels where the utilization factor is found to be greater than a selected cut-off utilization criterion (in the following example it is considered to be 2) are removed and the loads are redistributed and applied in the next iteration. The colour map of the panel usage factors is as follows:

Notation	Usage Factor
	0.2-0.4
	0.4-0.7
	>0.9
	>2
	Panel Removed

Figure 20. Colour map notation for Possible Case Scenarios

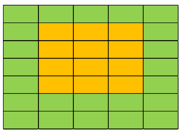
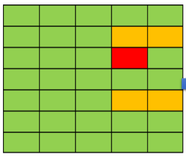
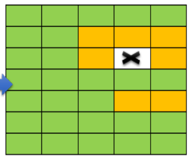
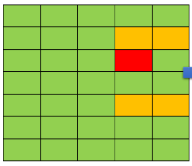
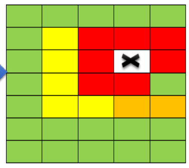
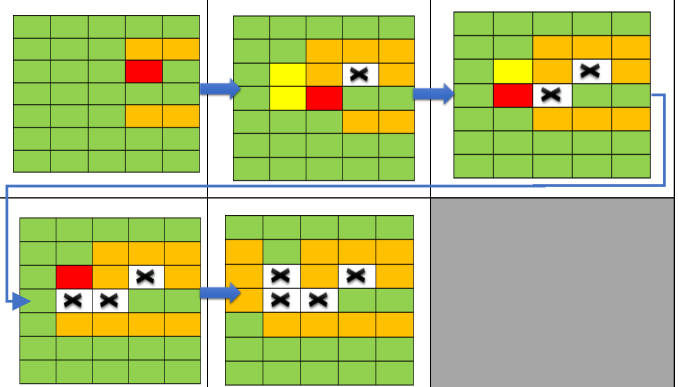
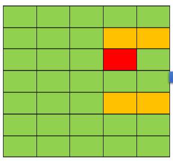
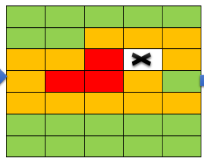
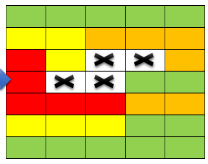
Case Scenarios	Iteration 1	Iteration 2	Iteration 3	Comments
Case 1				The inner panels have a slightly higher utilization factor -however the overall structure does not fail. The analysis stops after 1 iteration.
Case 2				More than 70% of the panels are highly utilized, and failure of these panels will lead to overall structure collapse.
Case 3				Only one panel exceeds the utilization criteria. On removal of this panel, the redistribution of the loads to the adjacent panels occurs. However Progressive collapse stops after one iteration.
Case 4				Only one panel exceeds the utilization criteria. Removal of this panel leads to the redistribution of loads to the adjacent panels. This redistribution of the load causes the adjaceent panels to collapse.
Case 5				Failure of one panel leads to adjacent panel collapse. However Progressive collapse stops after some iterations, and overall structure does not fail.
Case 6				Removal of the failed panel causes successive failing of adjacent panels over iterations and can lead to overall structural failure.

Table 1. Possible Case Scenarios

4.1.3 Analysis Flowchart

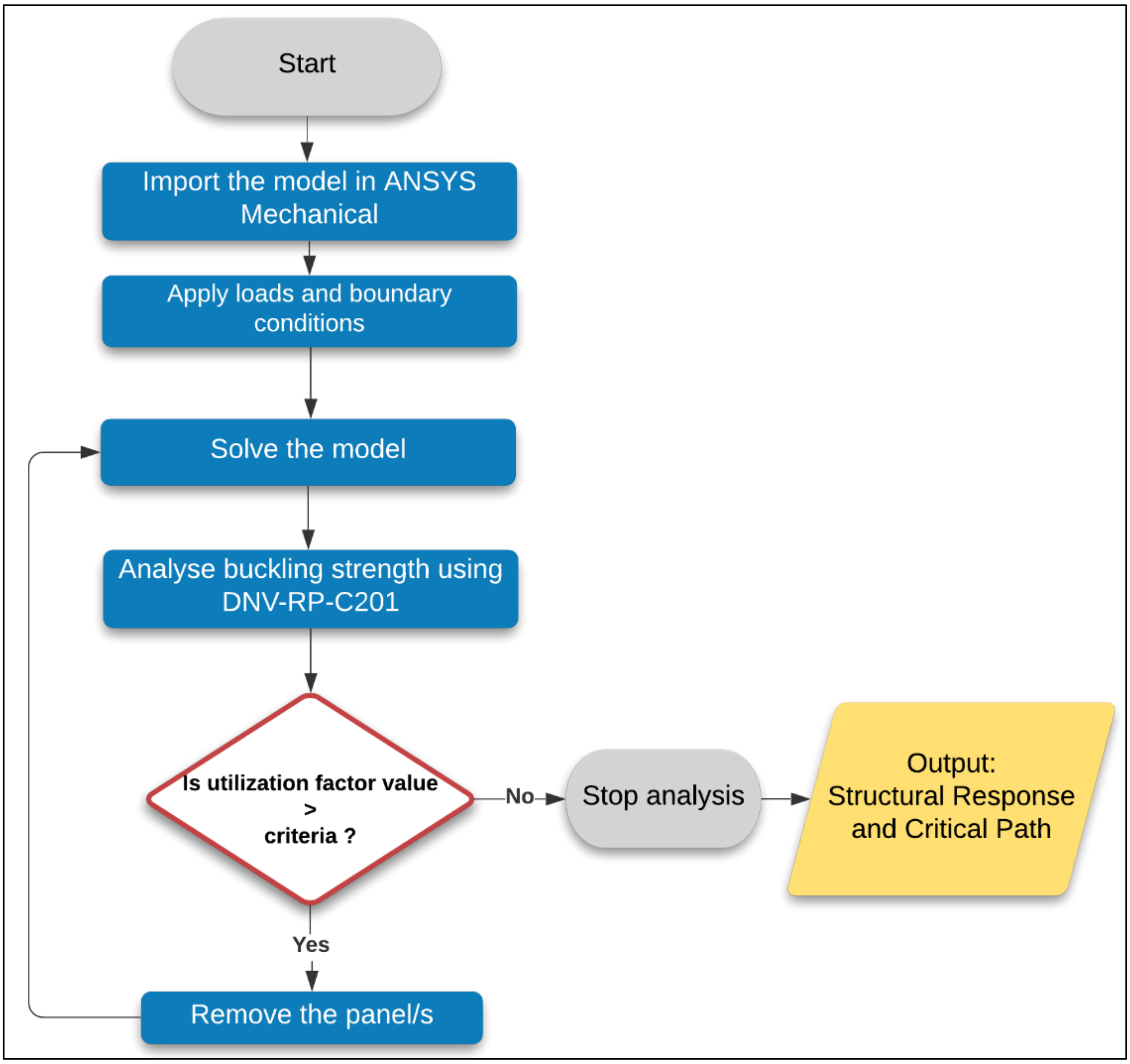


Figure 21. Redundancy Analysis Flow Chart

The redundancy analysis algorithm starts by importing the model of the cruise vessel in ANSYS Mechanical Academic Research and CFD, version 18.2, and applying the loads respective for the analysis. The evaluation of the progressive critical path is studied under hogging, sagging and a combination loading. In the case of hogging or sagging – the same load is applied at each iteration. In the combination loading case – hogging and sagging loads are applied alternatively at each iteration. The progressive collapse is analysed in both main structure, which concludes of 440 panels, and part of the structure at one time (e.g. one deck at specified z-coordinate) each considered at different iterative stages. After having concluded on the loads and boundary conditions, a buckling strength analysis is performed for the model and the utilization value specific to each panel is calculated Upon this, the utilization factor value needs to check the criteria imposed by the classification society rules (Section 4.4 Selection of Utilization Criteria, Page no. 46), respectively, if the utilization factor is higher than the limit imposed, another iteration will be performed, until the panel analysed will comply with the limitation. The panels exceeding the utilization criteria are assumed to have no load-carrying capacity and will be removed for the next iteration. The algorithm continues (analysis repeated) until the structural parts subjected to analysis are no longer exceeding the selected cut-off utilization criteria and then the progressive collapse analysis stops, providing as an outcome the structural response and critical path. This might represent the collapse of the structure in case of extreme loading condition.

4.2 Sample Model Description

A sample structural model was used to help develop the macros as shown in Figure 22. The model consists of 261 stiffened panels. The material is considered to be Linear Elastic Isotropic. ANSYS Mechanical APDL version 2020 R1, Academic Teaching introductory version was used for the sample model alone – as it is was accessible to work with. The sample model then meshed with a mesh size of 600 elements.

The Material used for the Sample model is normal steel with Young's Modulus E=$2.06*10^5$ MPa, Poisson's Ratio $\nu = 0.3$, and Density $\rho = 7.85 * 10^{-9}$ kg/mm^3. The implementation of this model was necessary to enable the testing of the algorithm. In order to check if the progressive collapse analysis is correct, a trial and error approach was carried out and numerous simulations were performed. The process requires a large amount of time and computation power to run an optimization iteration, verify if the results are in compliance with the imposed constraints, and if required, a modification would be appointed. Therefore, implementing the algorithm directly on the main model would have resulted in a time-consuming process, slowing the development of the code and the check of its validity.

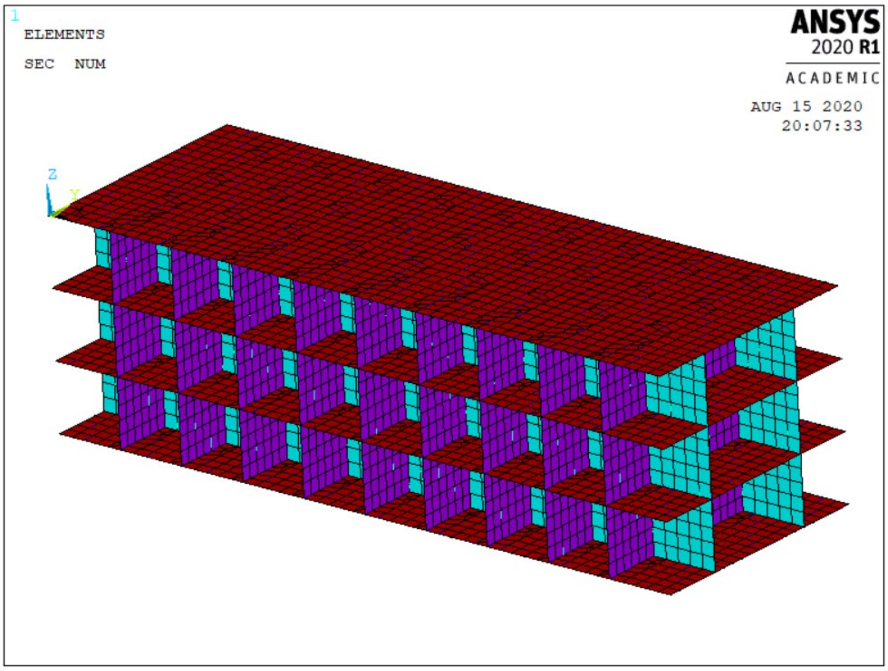


Figure 22. Sample Model

Loads and Boundary Conditions

For the boundary condition - it is assumed that the panel edges are simply supported i.e., with zero deflection and zero rotational restraints along all (four) edges with all edges kept straight. A simple load case was used, as it was purely for the development of the macro discussed.

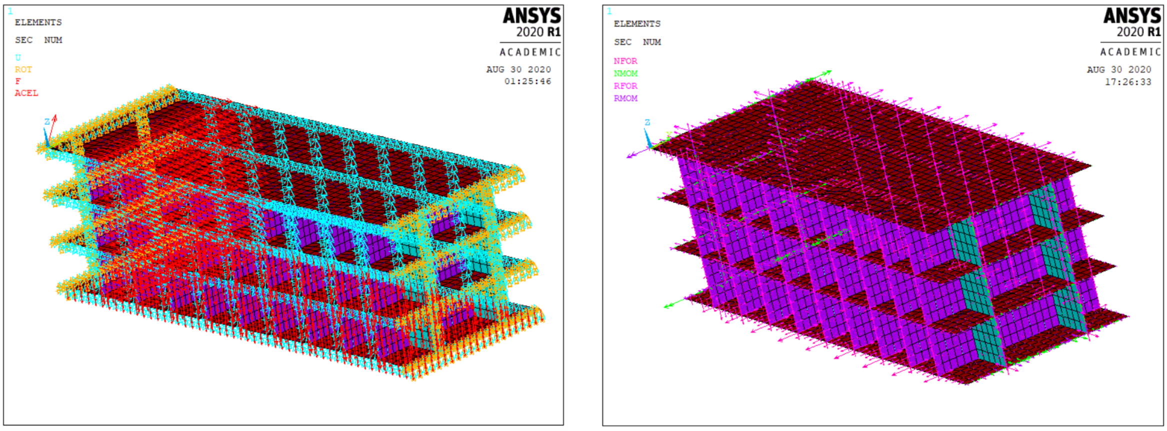


Figure 23. Boundary conditions and reactions on the sample model

Figure 23 shows the boundary conditions and reactions in the sample model. The boundary conditions are divided into two categories, essential boundary conditions, comprised of the fixed displacements and rotations, represented in the figure above by light blue for translations and yellow for rotations. The second type of boundary conditions are the reactions, respectively forces and accelerations, provided in red arrows. Regarding the loads acting on the structure, these are represented in the figure on the right by pink for the forces and their reactions and in green by the moments, having their reactions in a different shade of pink.

4.3 Development of the Macro

ANSYS Mechanical APDL Software has been used for performing strength simulations on the model through the implementation of macros (codes) written in Ansys Parametric Design Language. Various modular macros used in this thesis are developed for performing various tasks as listed below.

Macro	Function
inputpanel.mac	To import the coordinates from external CSV file in a table array using *tread command.
Mainloop.mac	To retrieve information related to stress, thickness, area, panel type, stiffener spacing etc from Ansys model.
BUCKLDNVRPC201.mac	User-defined function for estimating the utilization factor using the inputs from Mainloop.mac.
Analysis.mac	To find the panels with minimum and maximum usage factor.
Criticalpath.mac	3D annotation feature to show the critical progressive collapse path.
Usage_ plot.mac	To convert the utilization factor results from table array to etable for plotting.
3DAnno.mac	3D annotation feature to show the panels based on a criteria.
Panel_Removal.mac	To remove the failed panels based on utilization factor value or panel number.

Methodology

1. A list of panels with coordinates was created in an Excel CSV file in the following format:

Panel No	Xmin	Xmax	Ymin	Ymax	Zmin	Zmax

2. The coordinates were imported in a table array using the *inputpanel.mac* macro. Elements of the panels are selected based on the coordinates using *nsel* and *esel* commands.
3. The different structural parameters are loaded using the *Mainloop.mac* macro. Three different panel types were defined based on the orientation of the panels which are given below,

If $X_{min} = X_{max}$, then Panel type = Deck

If $Y_{min} = Y_{max}$, then Panel type = Longitudinal

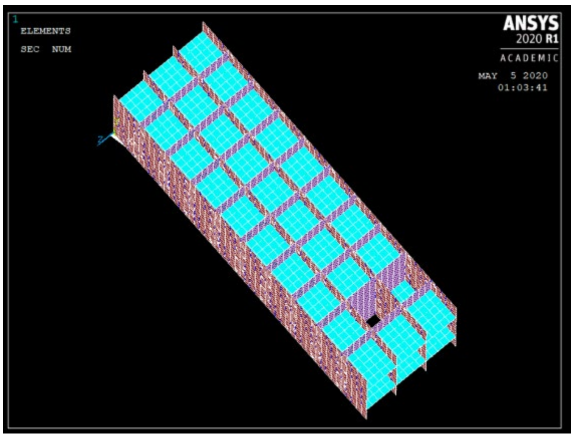
If $Z_{min} = Z_{max}$, then Panel type =Transverse

Based on panel types, the values of stress, stiffener spacing, the thickness of the plate, moment of inertia, area, length and breadth of the plates and stiffeners are retrieved and calculated using *get command.

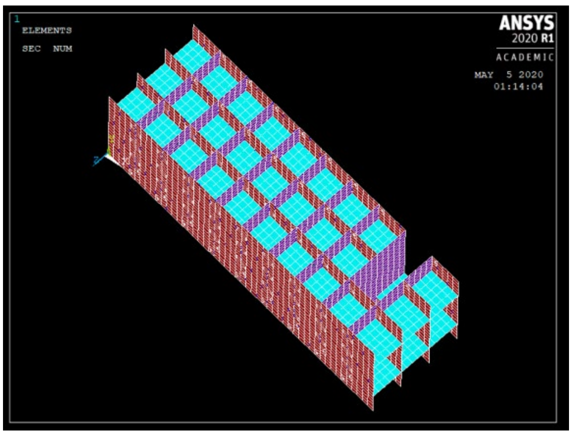
4. The user-defined function *BUCKLDNVRPC201.mac* is then used which takes inputs from Mainloop.mac macro, calculates the buckling strength using DNVGL RP-C201 rules and returns utilization factor values as the output.
5. The maximum and minimum utilization factor values were calculated and stored in an array using the *Analysis.mac* macro.
6. Panels were removed using *edele* and *ndele* commands based on utilization factor value using the *Panel_removal.mac* macro.
7. To visualize the critical progressive collapse path, the *Criticalpath.mac* with the /AN3D command is used.
8. Extracting the utilization factor details of each panel from the results array to etable in ANSYS for plotting the results is done using the final *Usage_Plot.mac* macro.

4.3.1 Sample Model – Progressive Collapse and Critical Path

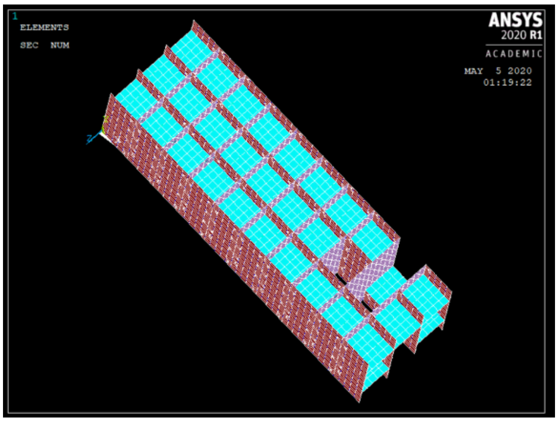
The progressive collapse in the case of the sample model, considering the cut-off utilization factor as 1 is illustrated below:



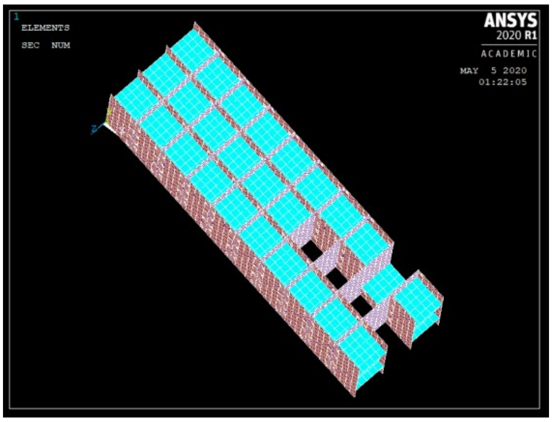
Panels failed after 1st iteration removed



Panels failed after 2nd iteration removed



Panels failed after 3rd iteration removed



Panels failed at the end of simulation. Progressive failure of panel stops.

Figure 24. Progressive Panel Failure for the Sample Model.

As it can be observed, the panels close to one end of the structure are utilized more, and in the successive iterations, the loading of the panels spreads towards panels near the centre of the structure increases and after 4 iterations the progressive failure of the panels stops. As most of the panels still maintain their integrity – the overall structure can be considered to be safe under the applied loads.

4.4 Cruise Vessel - Structure Description

After having developed the algorithm, using the Sample Model as a structural basis, a similar structure has been selected from the main model, to ease the implementation of the progressive analysis and strength assessment. For the simulations performed, two main loading cases are considered, sagging and hogging bending moment, providing extreme loading conditions, helpful in this type of analysis.

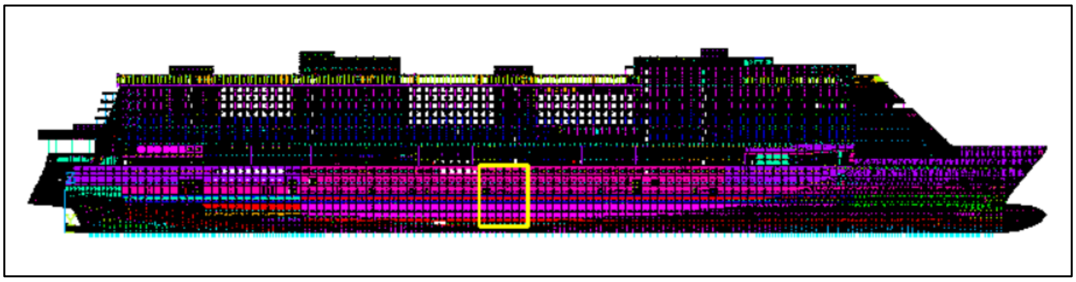


Figure 25 Cruise Vessel Structure - Side View

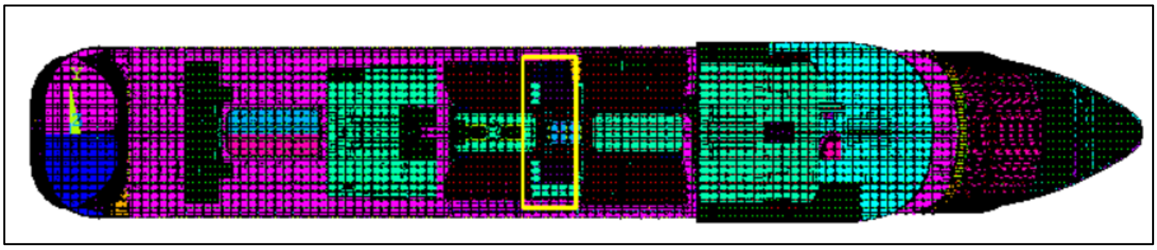


Figure 26 Cruise Vessel Structure - Top View

Since the model provided has a large number of finite elements, ANSYS Academic Research Mechanical and CFD, version 18.2 has been used for further implementation. The vessel used for analysis has a length of approximately 340m and an accommodation capacity of nearly 9500 passengers, crew and guests combined

The modelling and analysis of the complete hull girder using finite element method are computationally expensive. For this reason, the analysis is more conveniently performed on a section of the hull between two bulkheads, at the Midship section. (See Figure 25 and Figure 26)

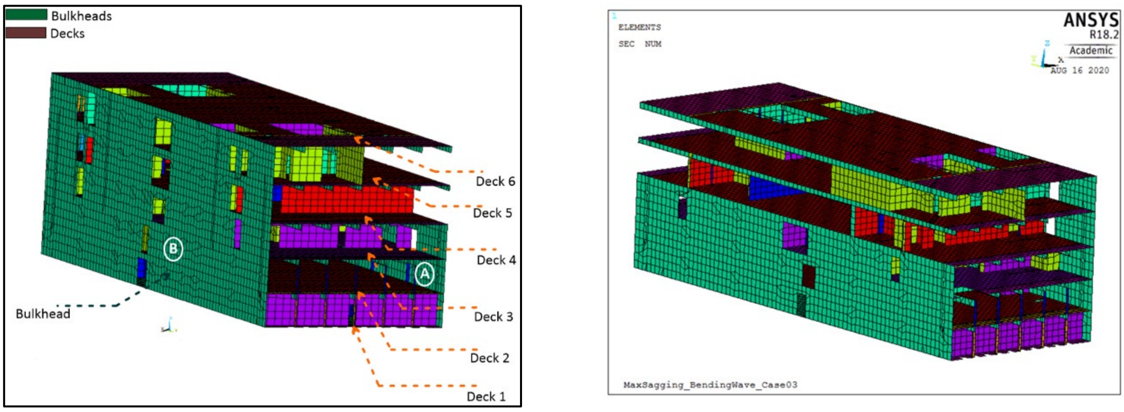


Figure 27 Main model

This section is chosen as will experience higher stress levels under different loads (i.e. hogging, sagging) than other parts of the vessel. A total of 440 stiffened panels spanning 2 bulkheads and 6 decks is considered, along with included longitudinal and transverse girders (Figure 27).

Deck Number	Distance from keel upwards (m)
Deck 1	7
Deck 2	9.85
Deck 3	12.75
Deck 4	15.75
Deck 5	19.05
Deck 6	22.35

Bulkhead Number	Distance from aft (m)
A	138.5
B	154.8

Table 2. Details of Parts of the Main Model

The panels between longitudinal and transverse girders are considered for the evaluation (see Figure 14). The material used in stiffened panels of merchant ship structures is usually mild or high tensile steel. **Steel grade A** is the common tensile strength steel and has good toughness properties and high strength, good corrosion resistance, and good processing and welding properties. It is optimum for construction of ship hull structure (Grade A shipbuilding steel plate, n.d.).

Parameter	Value	Unit
Tensile, yield strength	235	MPa
Ultimate Tensile Strength	400-520	MPa

Material 1: Properties steel grade A(*UR W11 Normal and Higher Strength Hull Structural Steels - Rev.9 May 2017*, n.d.)

Global Loads and Boundary Conditions

For the boundary conditions, it is assumed that the panel edges are simply supported with zero deflection and zero rotational restraints along all (four) edges with all edges kept straight. For steel stiffened plate structures, this assumption normally provides some pessimistic but adequate results

The evaluation of the progressive critical path is studied under hogging, sagging and a combination loading. The values of the loads used are extreme case load values, which cannot be disclosed due to confidentiality reasons. In the case of hogging or sagging – the same load is applied at each iteration. In the combination loading case – hogging and sagging loads are applied alternatively at each iteration.

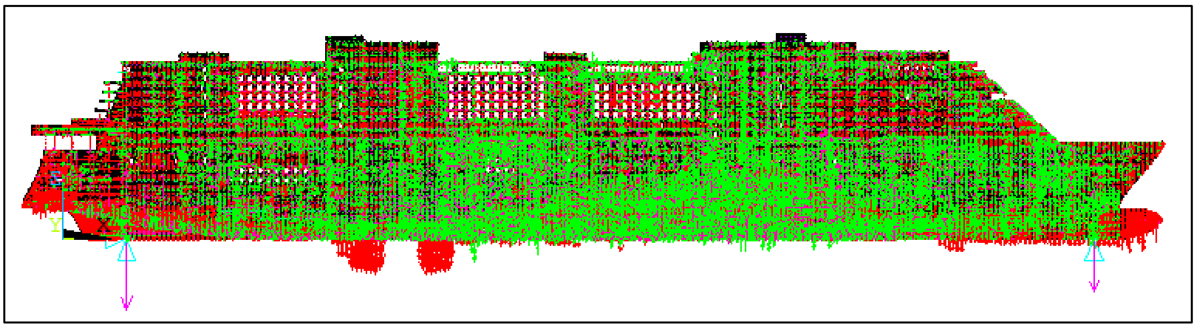


Figure 28. Global Load Case - Hogging

The colour code describing the boundary conditions and the loading of the main structure is as follows: in Figure 29 the red area is predominantly consistent with forces acting on the ship, whereas the green arrows represent moments. There are also constraints, respectively natural and essential boundary conditions, where the blue triangles depict the imposed constraints on displacements, and the pink arrows are the reactive forces.

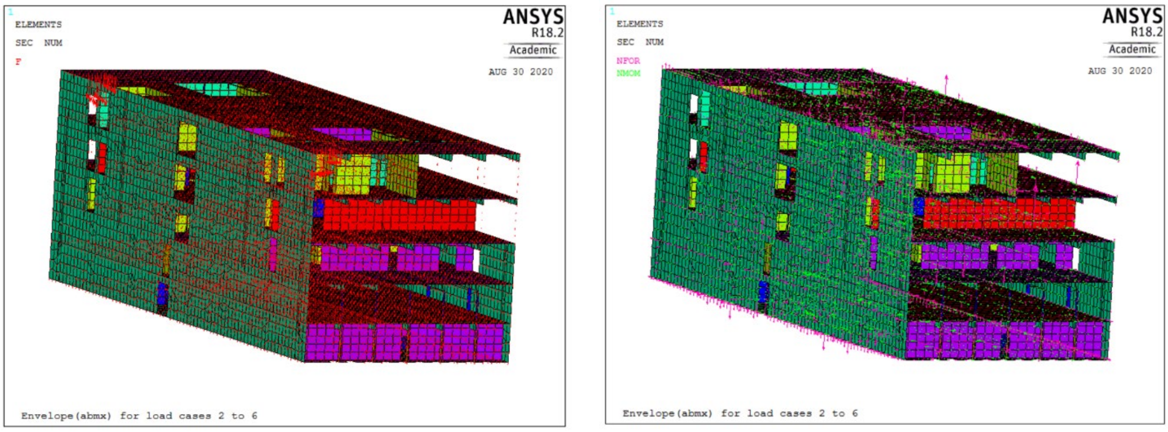


Figure 29. Boundary conditions and reactions on the Main model

Regarding the selected structure from the main model, boundary conditions are as explained previously for the full model, but at a different scale. Therefore, in Figure 29 we can visualize on the left the loads, respectively forces acting on the structure and on the right the moments and forces constraining the model at a local level.

In addition to the main model-specific components of structure in the main model are studied individually as well. The list of analysis carried out are as follows:

Different Loading Conditions analyzed for the Main Model and substructures		
Part	Total no of Panels	Loading Condition
Full Model	440	Hogging
Full Model	440	
Full Model	440	
Bulkhead A	49	
Deck at 7m	42	
Deck at 2m	45	
Deck at 22.35m	44	
Full Model	440	Sagging
Deck at 22.35m	44	
Full Model	440	Load Combination

Table 3. List of all analysis carried out

Selection of Utilization Criteria

As per DNV Regulations, a panel is considered to have failed when its utilization factor is calculated to be greater than 1. However, in this thesis, as the applied global loads cannot be varied, the utilization criteria cut-off is to be modified in order to study the progressive critical failure path when increasing number of panels fail. The selection of this cut-off utilization criteria value is done by analysing the number of panels that fail at the end of the 1st iteration. It is to be noted that these cut-off values differ for the different loading conditions.

Consider the case of the bulkhead in hogging, on evaluating the utilization factor of the panels after the first iteration, the following plot was obtained

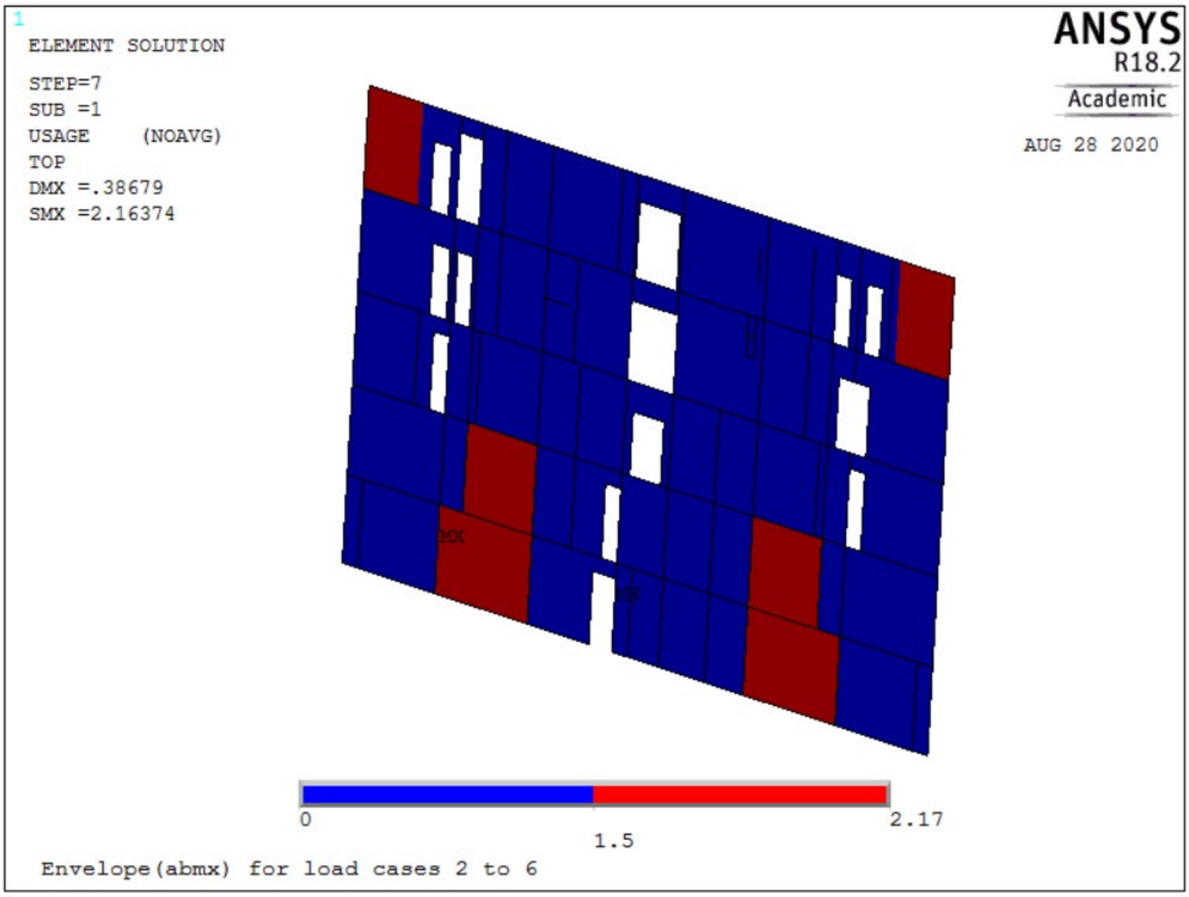


Figure 30. Usage Plot of Bulkhead in Hogging

As it can be seen 6 panels in the bulkhead have a utilization factor greater than 1.5 – therefore to evaluate the scenario where 12% of the panels fail at the first iteration – the cut-off utilization criteria of 1.5 was selected. In this fashion different cut-off utilization criteria were selected for the main model and its different substructures in order to analyse the progressive failure of the structure when increasing fractions of the structure fail after the first iteration.

Analysis	Part	Total no of Panels	Load Case	Cut-off Utilization value	Percentage of Panels failed in the first iteration
1	Bulkhead	49	Hogging	1.50	12%
2	Deck at 22.35m	44	Sagging	0.08	32%
3	Full Model	440	Hogging	0.03	70%
4	Full Model	440	Sagging	0.03	70%
5	Full Model	440	Load Combination	0.10	37%
6	Full Model	440	Load Combination	0.06	50%
7	Deck at 22.35m	44	Hogging	0.02	7%
8	Deck at 2m	45	Hogging	0.05	58%
9	Full Model	440	Hogging	0.10	37%
10	Full Model	440	Hogging	0.06	50%
11	Deck at 7m	42	Hogging	0.06	31%
12	Deck at 22.35m	44	Sagging	0.15	14%

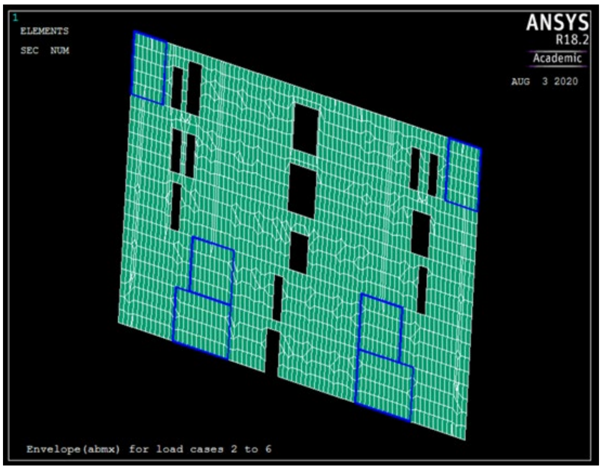
Table 4. List of all analysis carried out and corresponding selected cut-off utilization factor

Reducing the utilization factor as explained above is considered to be equivalent to increasing the loads, and therefore it is to be kept in mind that panel failure refers to the utilization factor of the respective panel being greater than the cut-off criteria. The highlighted cases in Table 4 are presented in detail in the thesis. Other cases analyses are placed at Appendix A1.

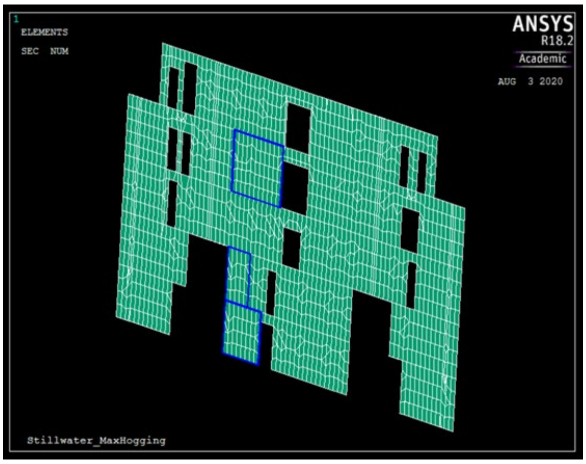
5 RESULTS

5.1 Analysis 1 - Hogging Load on the Bulkhead

In order to demonstrate the results clearly – the simple case of the progressive panel failure in the case of hogging load on the bulkhead at one end of the structure considered is given below. On selecting the utilization criteria at 1.5, it was found that 6 of the total 49 panels in the bulkhead fail after the first iteration. The simulation was extended to see if progressive panel failure stopped before too many panels were compromised. After 4 iterations, progressive panel failure stopped, after a total of 10 panels have failed and been removed.

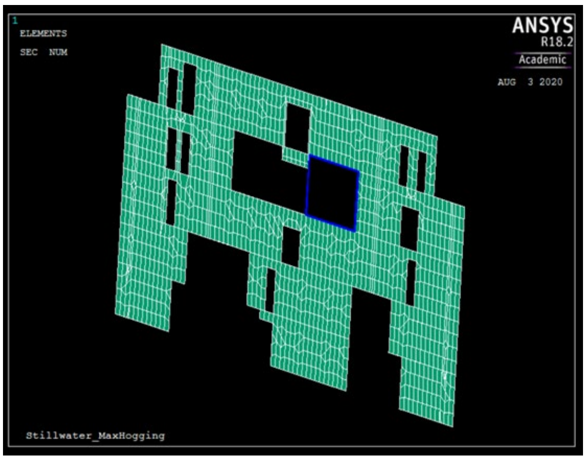


6 Panels failed after 1st iteration



3 Panels failed after 2nd iteration

1 Panel failed after 3rd iteration



Panels failed at the end of simulation. Progressive failure of panel stops.

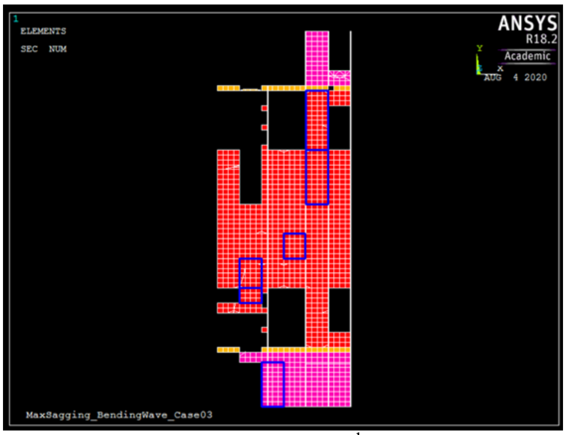
Figure 31. Progressive Panel Failure for a bulkhead of the Cruise Vessel Model under hogging load, utilization criteria 1.5

At the end of the analysis, only approx. 20% of the 49 panels have failed, thereby it can be concluded that 1/5th panels on this bulkhead have failed under the hogging load – but the overall bulkhead does not fail in this scenario.

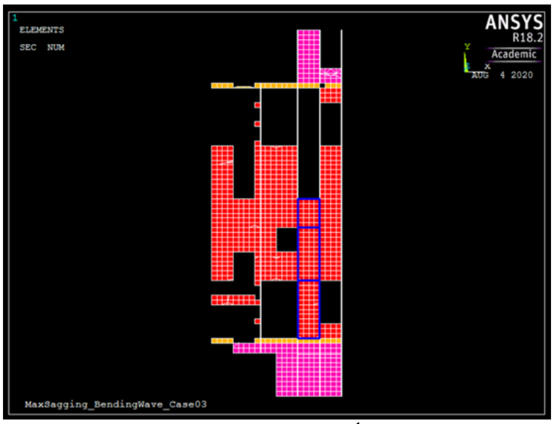
5.2 Analysis 2 - Sagging Load on the Deck at 22.35m

The progressive panel failure in the case of sagging load on the Deck at 22.3 m at the top of the Cruise vessel selected structural area is considered. Under sagging load as the top deck is under compression, therefore panels here are prone to buckling failure. The selected utilization criteria is 0.08, and after one iteration 14 out of the 44 deck panels fail, at the edges of the vessel athwartship. The simulation was again extended to see if progressive panel failure stopped before too many panels were compromised. After 4 iterations, progressive panel failure stops, however at the end of the simulation 24 panels fail, which is 54.5%. The failure of the panels starts near the sides of the vessel athwartship and progressively the panels near the centreline.

14 Panels failed after 1st iteration

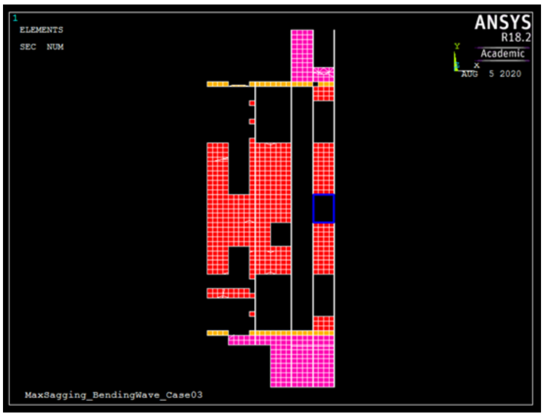


6 Panels failed after 2nd iteration



3 Panels failed after 3rd iteration

1 Panel failed after 4th iteration

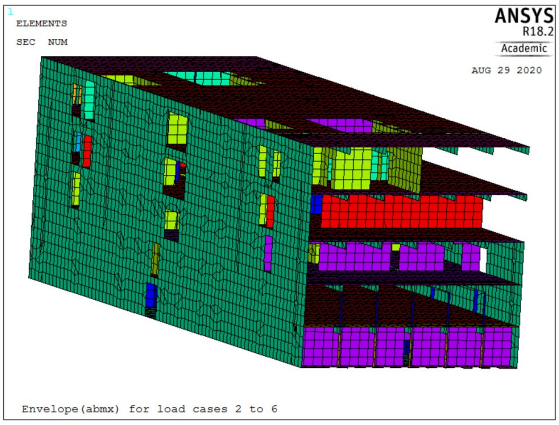


Panels failed at the end of simulation.
Progressive failure of panel stops.

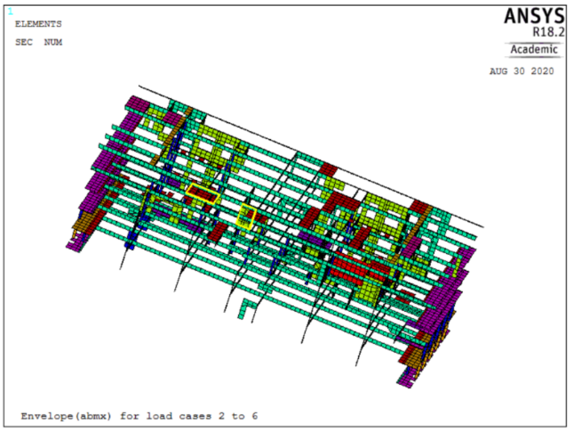
Figure 32. Progressive Panel Failure for a Deck at 22.3 of the Main Model under sagging load, utilization criteria 0.08

5.3 Analysis 3 – Hogging Load on the Main Model

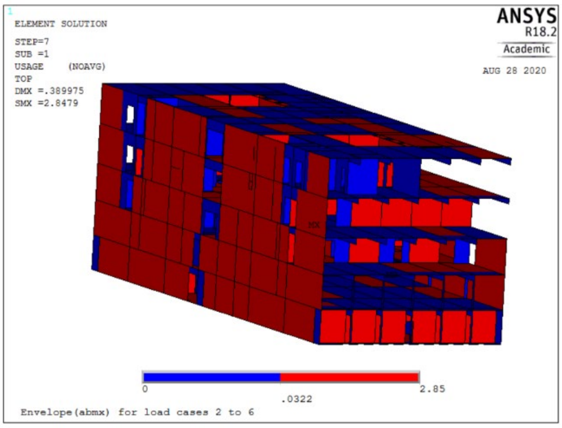
The progressive panel failure in the case of hogging load on the overall Main Model is now considered. The selected utilization criteria is 0.03, and after the first iteration, 70% of the panels fail. The reason for selecting such a low utilization factor – was to verify if progressive collapse would stop before all the panels fail. At the end of the analysis, all but 2 panels fail out of the total 440 panels on the structure. Therefore, it can be seen that if a high number of panels fail in the first iteration – the whole structure subsequently fails.



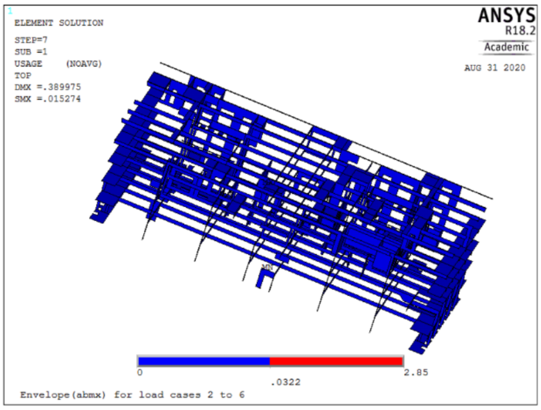
Main Model: Initial State



Main Model: Final Iteration Result



Main Model: Usage plot after 1st Iteration



Main Model – After Final Iteration

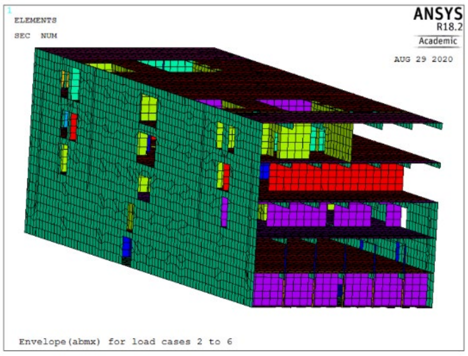
Figure 33. Progressive Panel Failure for the Main Model in Hogging – Usage Factor set at 0.03

In the image below – the various girders and other panels are visualized as well, the panels that have not failed at the end of the analysis are highlighted in yellow colour.

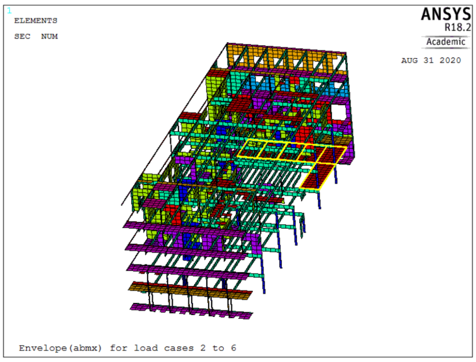
Although 2 panels do not fail at the end of the analysis – it does **not** imply that the entire load is carried by these panes without failing. This is because through the iterations, as the panels are removed a part of the load is also successively removed.

5.4 Analysis 4 - Sagging Load on the Main Model

Similar to the previous analysis - the progressive panel failure is now analysed in the case of sagging load, with the selected utilization criteria as 0.03. After the first iteration, 70% of the panels fail in this case as well. At the end of the analysis, all but 6 panels fail out of the total 440 panels on the structure. Therefore, it can be concluded that if a high number of panels fail in the first iteration, greater than 70%, the whole structure subsequently fails – irrespective of the loading case (hogging or sagging).

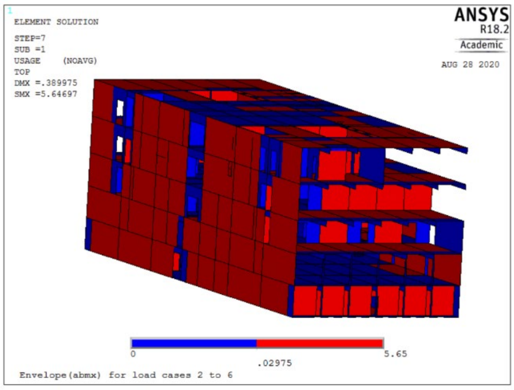


Main Model: Initial State

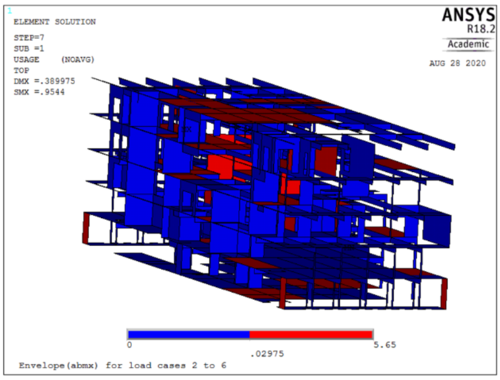


Main Model: Final Iteration Result

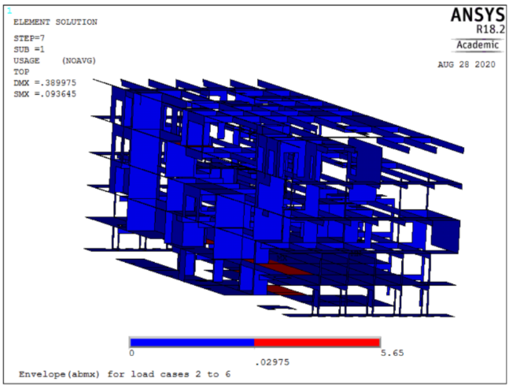
Figure 34. Progressive Panel Failure for the Main Model in Sagging – Usage Factor set at 0.03



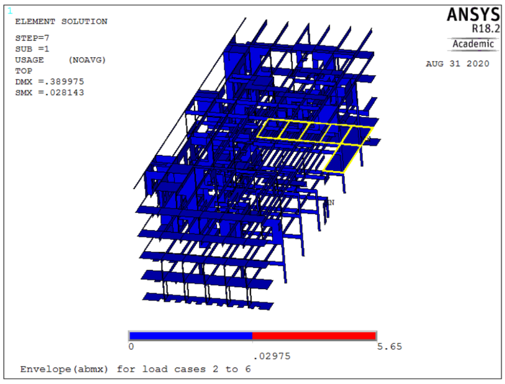
Usage Plot: First Iteration



Usage Plot: Second Iteration



Usage Plot: Third Iteration

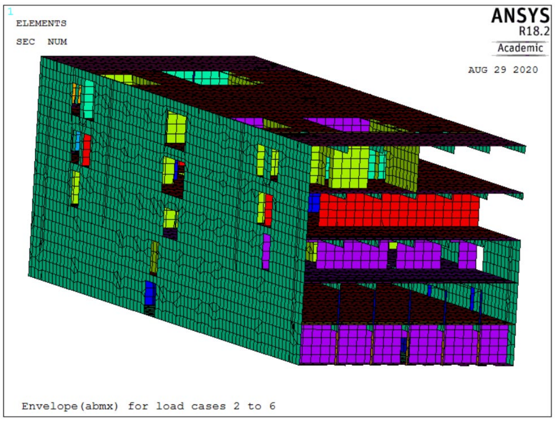


Usage Plot: Fourth Iteration

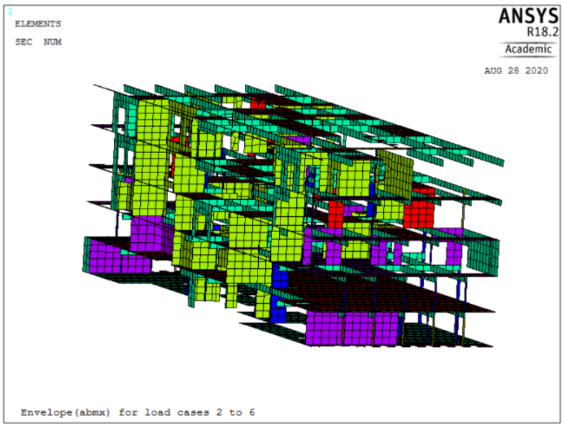
Figure 35. Usage Factor Plots over different iterations for the Main Model in Sagging – Usage Factor set at 0.03

5.5 Analysis 5 - Load Combination on the Main Model

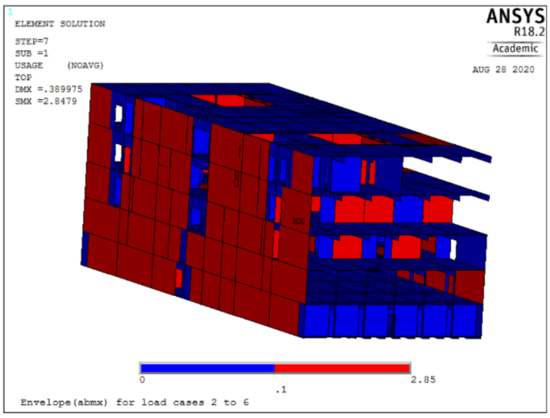
Alternating Hogging and Sagging Loads are applied to the different iterations of the analysis, in order to evaluate the progressive panel failure in this extreme case scenario. the selected utilization criteria is 0.1. After the first iteration, 37% of the panels fail and at the end of the analysis, 73% of the panels have failed.



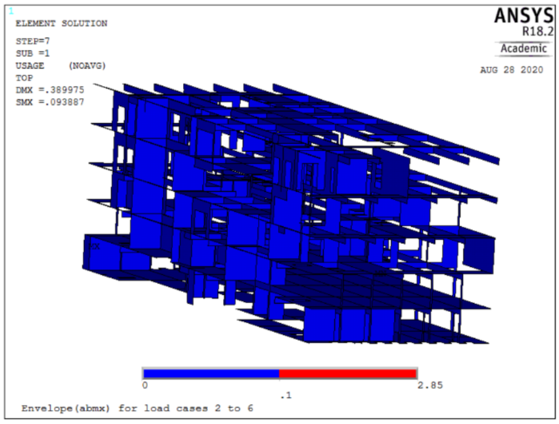
Main Model: Initial State



Main Model: Final Iteration Result



Main Model: Usage plot after 1st Iteration

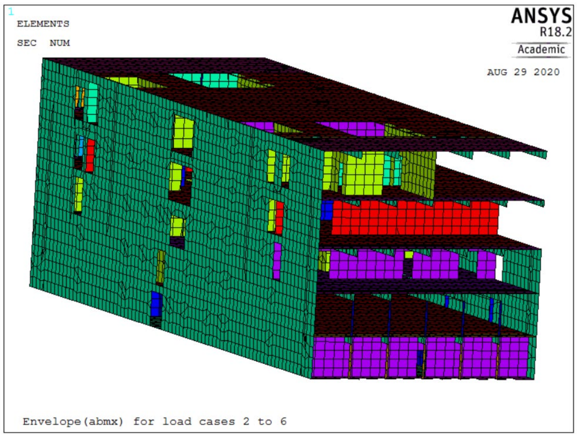


Main Model – Usage Plot After Final Iteration

Figure 36. Progressive Panel Failure (above) and Usage Factor Plots (below) over different iterations for the Main Model in Load Combination – Usage Factor set at 0.1

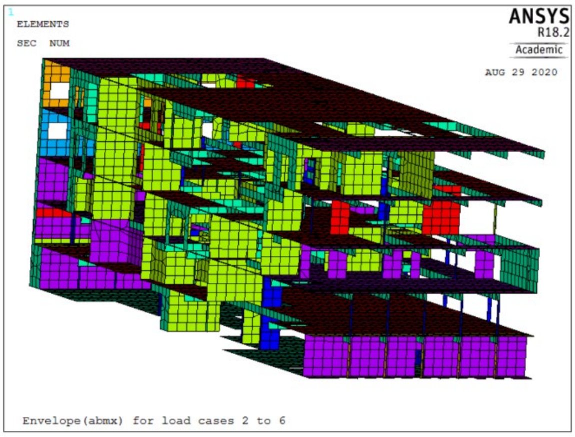
Now this case scenario is compared to the results obtained in the event of a pure hogging load, with the same usage factor of 0.1

Pure Hogging Load



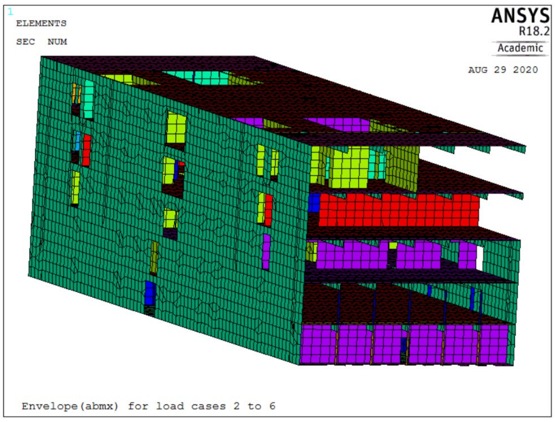
Main Model: Initial State

Pure Hogging Load



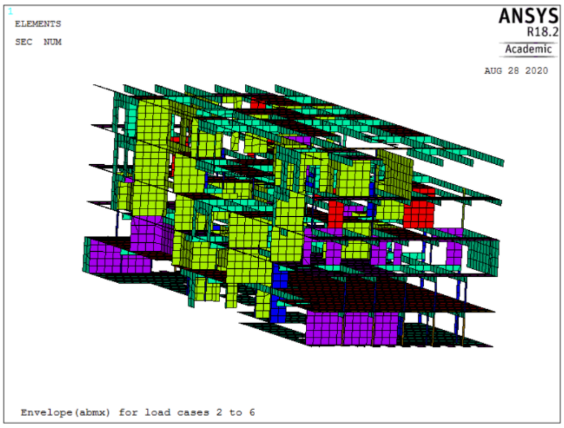
Main Model: Final Iteration Result

Load Combination – Alternating Hogging and Sagging Load



Main Model: Initial State

Load Combination – Alternating Hogging and Sagging Load

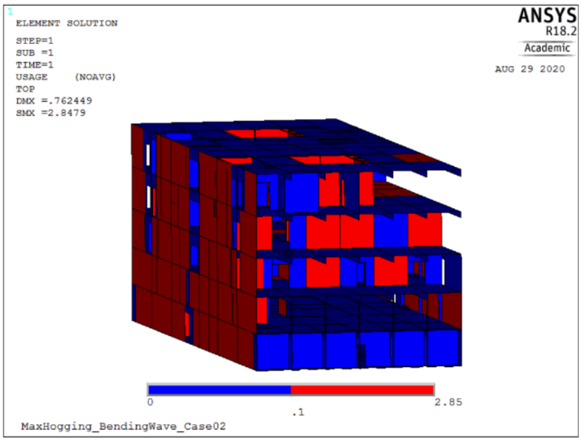


Main Model: Final Iteration Result

Figure 37. Progressive Panel Failure for the Main Model in Pure Hogging (above) and Load Combination (below) both - Usage Factor set at 0.1

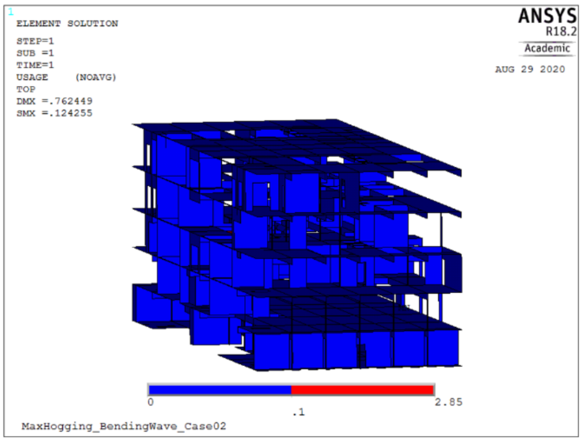
It can be seen that panels at the top and bottom fail simultaneously in the case of the load combination, whereas in the case of pure hogging, the panels on the top remain mostly intact, as expected.

Pure Hogging Load



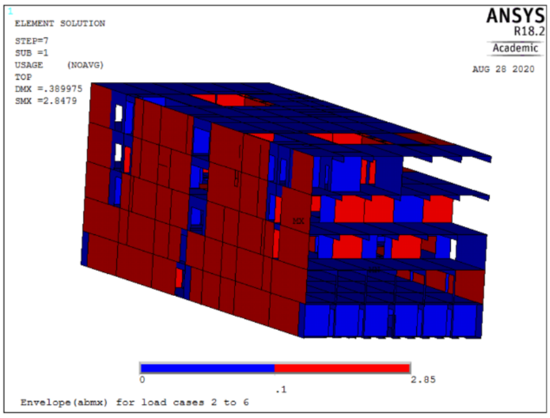
Main Model: Usage plot after 1st Iteration

Pure Hogging Load



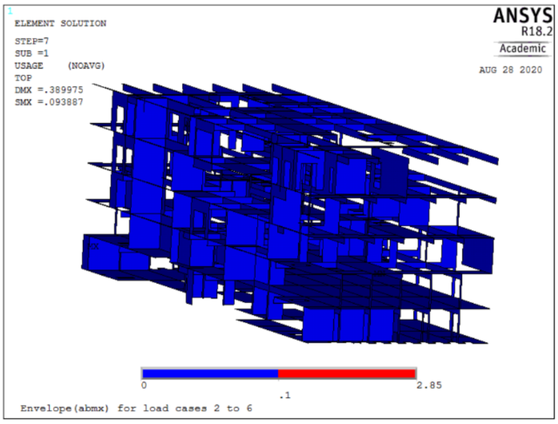
Main Model – Usage Plot after Final Iteration

Load Combination – Alternating Hogging and Sagging Load



Main Model: Usage plot after 1st Iteration

Load Combination – Alternating Hogging and Sagging Load



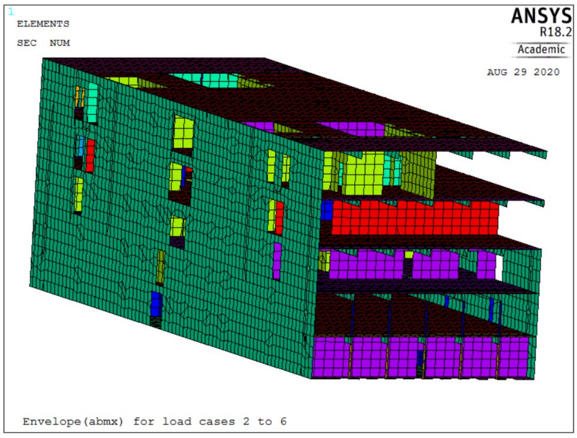
Main Model – Usage Plot after Final Iteration

Figure 38. Usage Plot for the Main Model in Pure Hogging (above) and Load Combination (below) both - Usage Factor set at 0.1

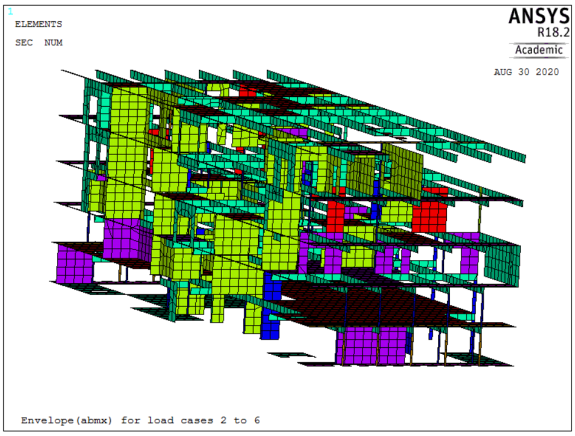
As it can be seen that the load combination results in a higher number of panels failing before the analysis stops. At the end of the analysis in the case of pure hogging, 51.4% panels fail – whereas in the load combination 72.5% of the panels fail before the analysis stops. This is a significantly higher number of panels. Therefore, it can be concluded that this load combination is a more critical loading scenario than pure hogging load, particularly for buckling.

5.6 Analysis 6 - Load Combination on the Main Model

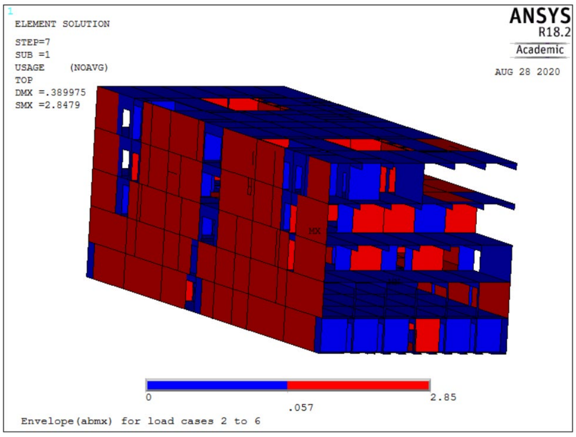
Alternating Hogging and Sagging Loads as previously mentioned in the previous analysis are applied again, and this time the cut-off utilization criteria is set lower at 0.06. This analysis is done to see if the progressive failure of the panels differs from the previous case scenario with a higher cut-off utilization criteria. After the first iteration, 50% of the panels fail and at the end of the analysis, 81% of the panels fail.



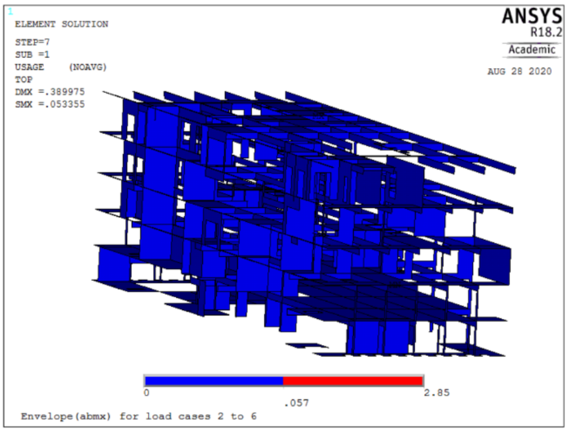
Main Model: Usage plot after 1st Iteration



Main Model: Final Iteration Result



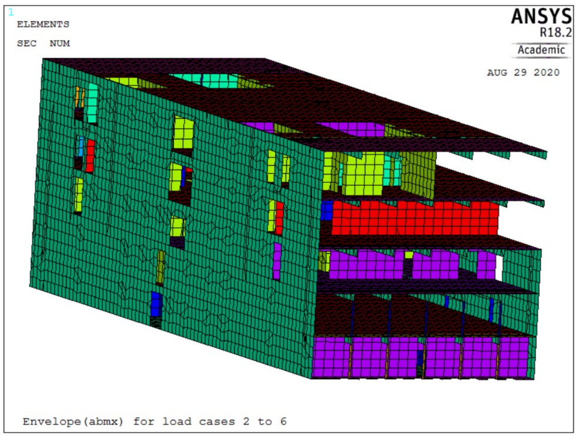
Main Model: Usage plot after 1st Iteration



Main Model: Usage plot after Final Iteration

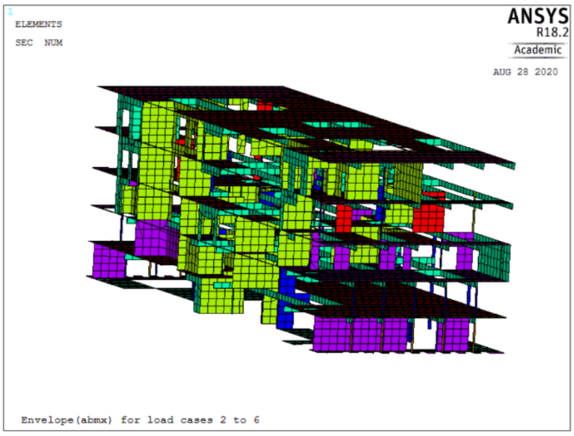
Figure 39. Progressive Panel Failure (above) and Usage Factor Plots (below) over different iterations for the Main Model in Load Combination – Usage Factor set at 0.1

Pure Hogging Load



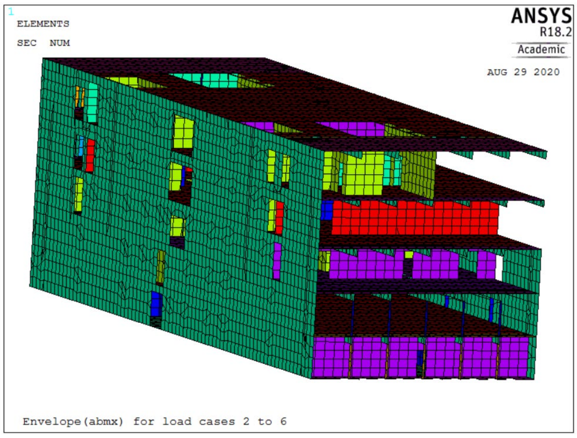
Main Model: Initial State

Pure Hogging Load



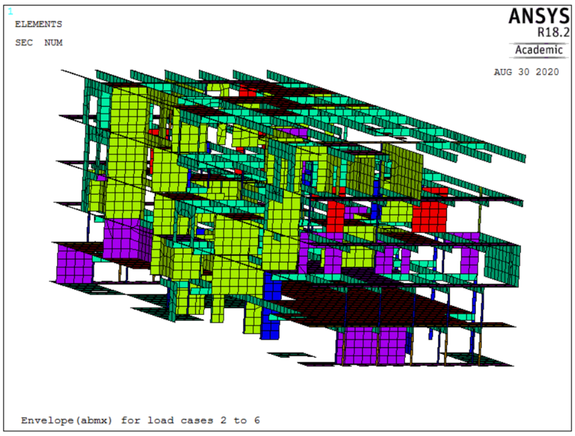
Main Model: Final Iteration Result

Load Combination – Alternating Hogging and Sagging Load



Main Model: Initial State

Load Combination – Alternating Hogging and Sagging Load

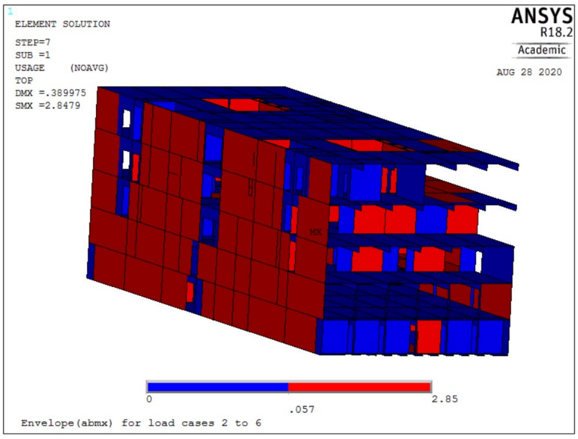


Main Model: Final Iteration Result

Figure 40. Progressive Panel Failure for the Main Model in Pure Hogging (above) and Load Combination (below) both - Usage Factor set at 0.06

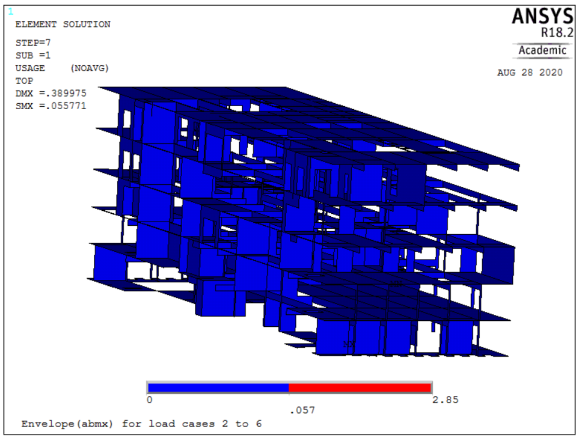
It can be once again observed that panels at the top and bottom fail simultaneously in the case of the load combination, whereas in the case of pure hogging, the panels on the top remain mostly intact.

Pure Hogging Load

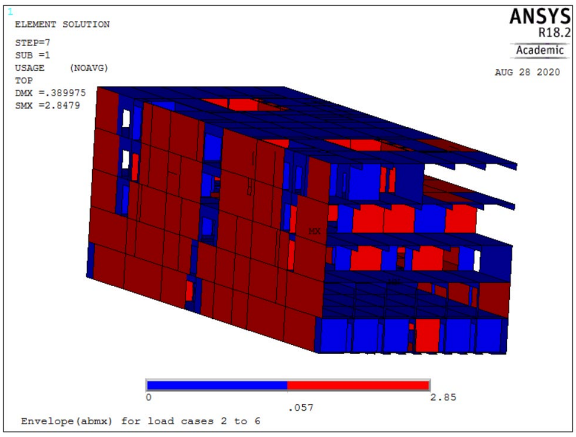


Main Model: Usage plot after 1st Iteration
Load Combination – Alternating Hogging and Sagging Load

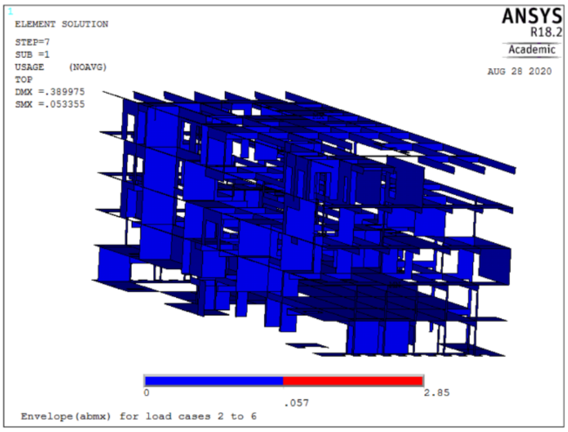
Pure Hogging Load



Main Model – Usage Plot after Final Iteration
Load Combination – Alternating Hogging and Sagging Load



Main Model: Usage plot after 1st Iteration



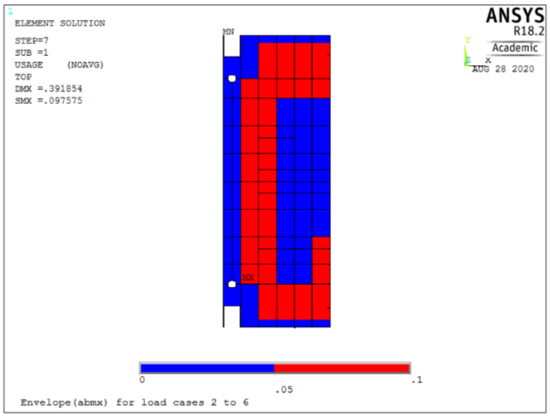
Main Model – Usage Plot after Final Iteration

Figure 41. Usage Plot for the Main Model in Pure Hogging (above) and Load Combination (below) both - Usage Factor set at 0.06

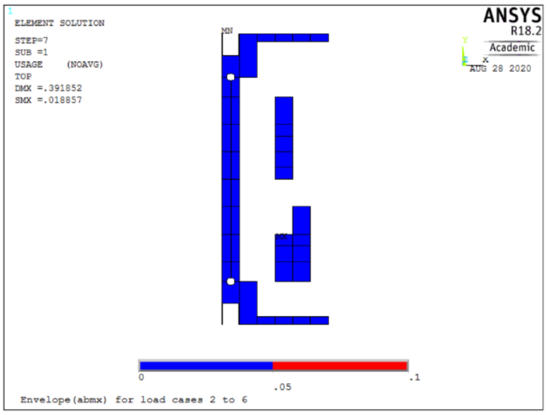
Once again, the load combination results in a higher number of panels failing before the analysis stops. At the end of the analysis in the case of pure hogging, 67.7% panels fail – whereas in the load combination 81% of the panels fail before the analysis stops. This is a significantly higher number of panels. Therefore, it can be confirmed that this load combination is a more critical loading scenario than pure hogging load, particularly for buckling.

5.7 Analysis 7 & 8 - Hogging Load on Deck at 22.35 m and Deck at 2m.

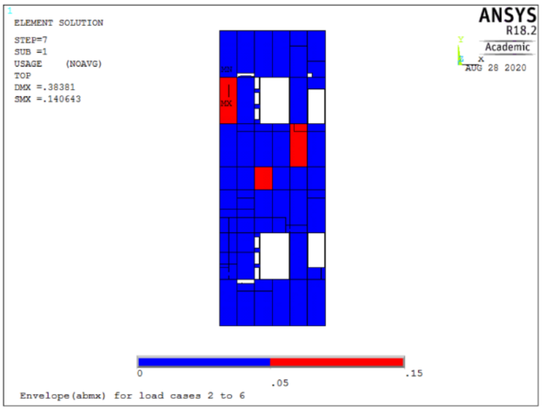
The analysis of these two decks are considered together as they are form the top and bottom deck of the main model. The behaviour of these two decks are considered individually in hogging. The usage factors were selected as 0.05 for the bottom Deck as 2 m and as 0.02 for the Deck at 22.3m. The reason for this difference in the selected usage factors is that selecting a usage factor greater 0.02 results in no panels failing for the top Deck. As the usage factors are close, the obtained results are considered to be comparable.



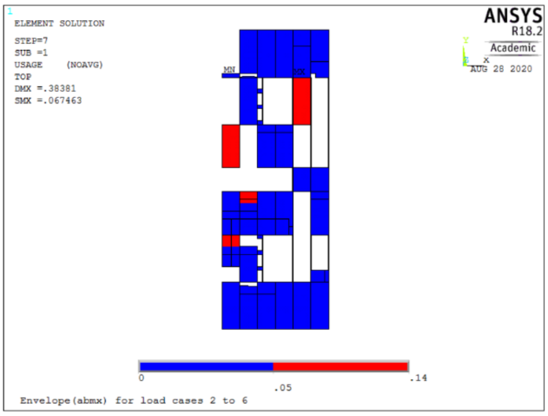
Main Model: Usage plot after 1st Iteration



Main Model – Usage Plot after Final Iteration



Main Model: Usage plot after 1st Iteration



Main Model – Usage Plot after Final Iteration

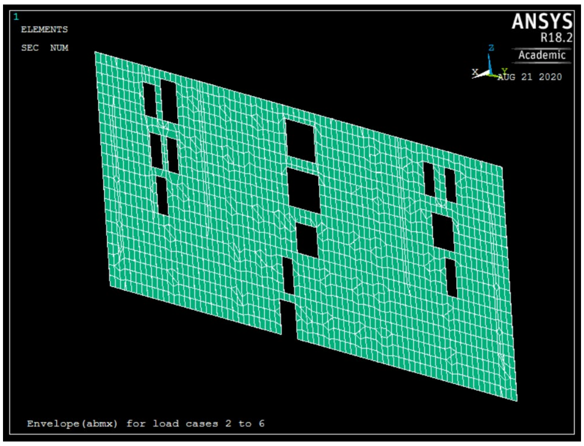
Figure 42. Usage Factor Plot for Deck at 2m (above) and 22.3 m (below)

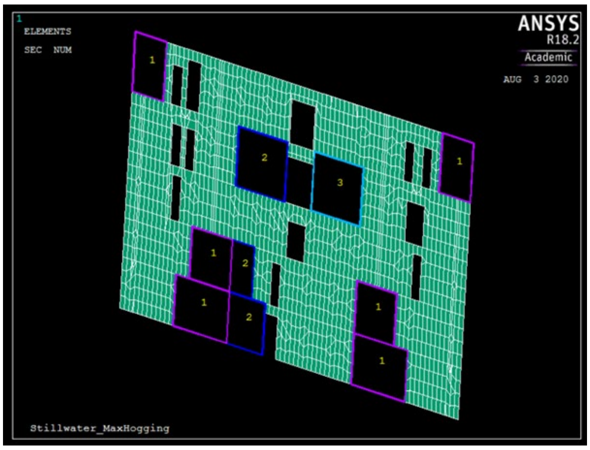
It can be observed that the for the Deck at 2 m, the number of panels that fail at the first step is much higher than the panels at the higher Deck. This is because as expected under a hogging load, lower deck panels are subject to compression thereby fail by buckling much easier compared to panels at higher decks. At the end of the analysis – 82.2% of the panels fail on the lower deck – whereas only 31.8 % of the panels fail at the top deck.

5.8 Inferences

1. Progressive Critical Failure Path

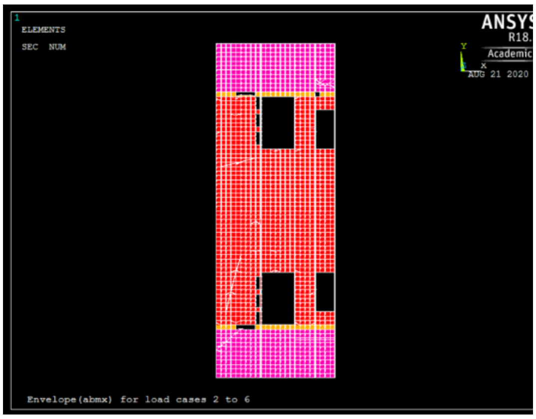
The progressive critical failure path in the case of the hogging load on the bulkhead is illustrated below.

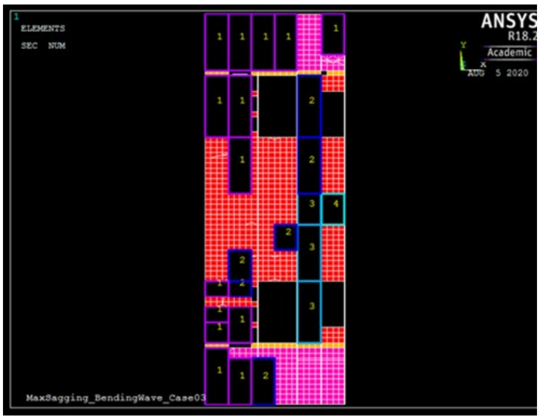




It can be seen that six panels exceeded the criteria. On removing these panels, the load shifts to the inner panels near the centre of the structure. The critical failure path has a level of symmetry about the Z-axis, the discrepancies can be attributed to unsymmetrical cut-outs in the bulkhead.

For the case of the higher deck under sagging, the progressive critical failure path is as follows:

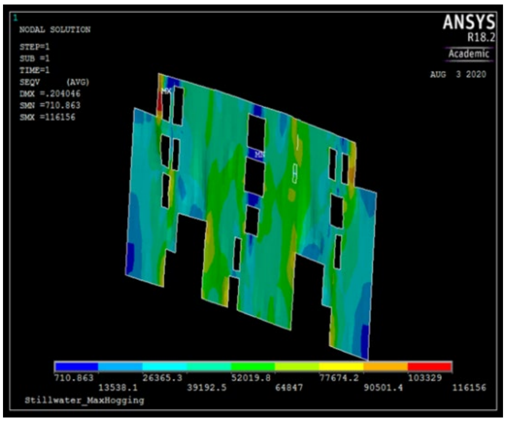




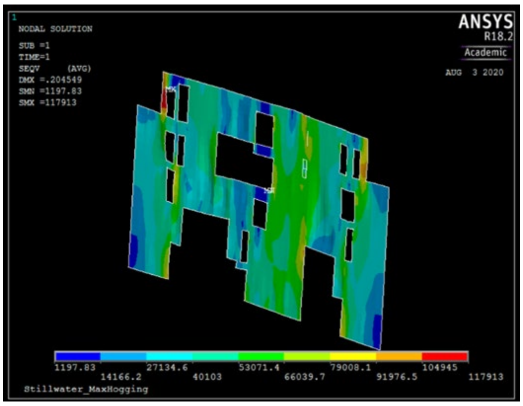
It can be seen that the outer panels fail easily, due to the boundary condition effect. Over the iterations, the load redistributes itself to the inner panels. Interestingly after the aft panels fail and are removed, in the subsequent iterations the forward panels near the deck cut-outs begin failing. At the last iteration, the panel at midship in the forward alone fails – and this suggests that this panel is critical to stopping progressive failure.

2. Stress values in the structure

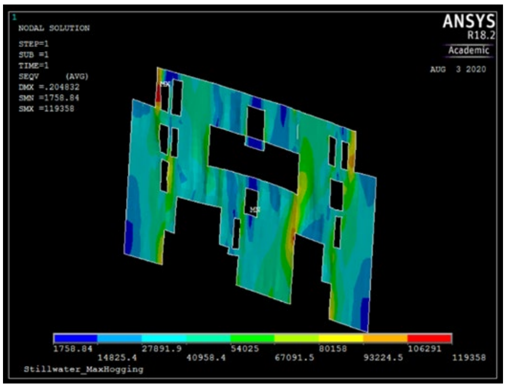
In the case of the bulkhead under hogging, the stress values developed in the structure after 1 iteration is studied. It was observed that progressive removal of panels causes the von Mises stresses in the adjacent panel structure to increase significantly. Understandably in real case scenarios – the stress values will not reach such high values as the panels that fail in buckling would still have some load carrying capacity and are not 'removed'.



von Mises stress on the bulkhead after the 1st iteration



von Mises stress on the bulkhead after the 2nd iteration



von Mises stress at the end of the analysis

Figure 43. von Mises stress developed on the Bulkhead in Hogging

3. Analysis of the contribution of stiffeners in the structure

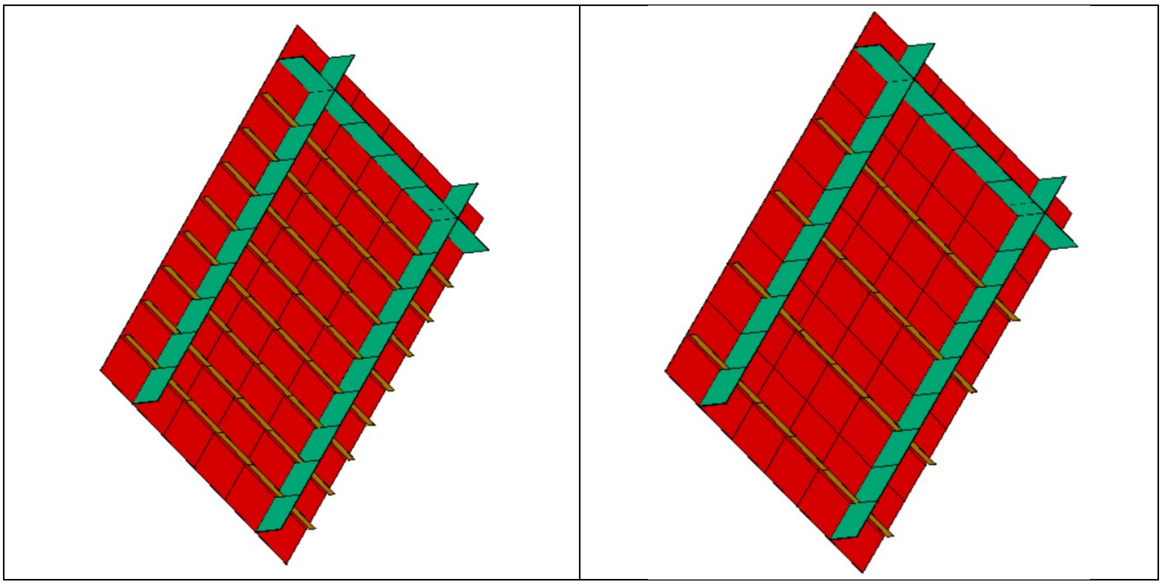


Figure 44. Panel with Stiffener spacing = 0.6 m (Left) and 1.2 m (Right)

The effect of varying the stiffener spacing on a simple panel is considered, for the hogging load. Initially, a lower spacing (thereby a greater number of stiffeners) of 0.6m is considered. The utilization factor of this panel is then calculated to be 0.13. On increasing the stiffener spacing (and reducing the number of stiffeners) – the utilization factor of the panel now increases to 0.25. It can be seen that increasing the number of longitudinal stiffeners will increase the load-carrying efficiency of the plate material, which is a function of buckling.

In order for this plate to have the same usage factor with half the stiffeners however, the plate thickness of the panel needs to be increased from 6mm to 7.5mm. This aspect is useful as the weight of the structure can be reduced greatly by increasing the number of stiffeners and decreasing the plate thickness.

4. Varying Deck Thickness

The influence on the thickness of the deck plating is considered for the case of the Deck at 7 m. Initially, the plate thickness of the deck is 8mm. The utilization factor obtained after the first iteration is obtained. The deck thickness is then reduced by half to 4 mm observe if the utilization factor varies.

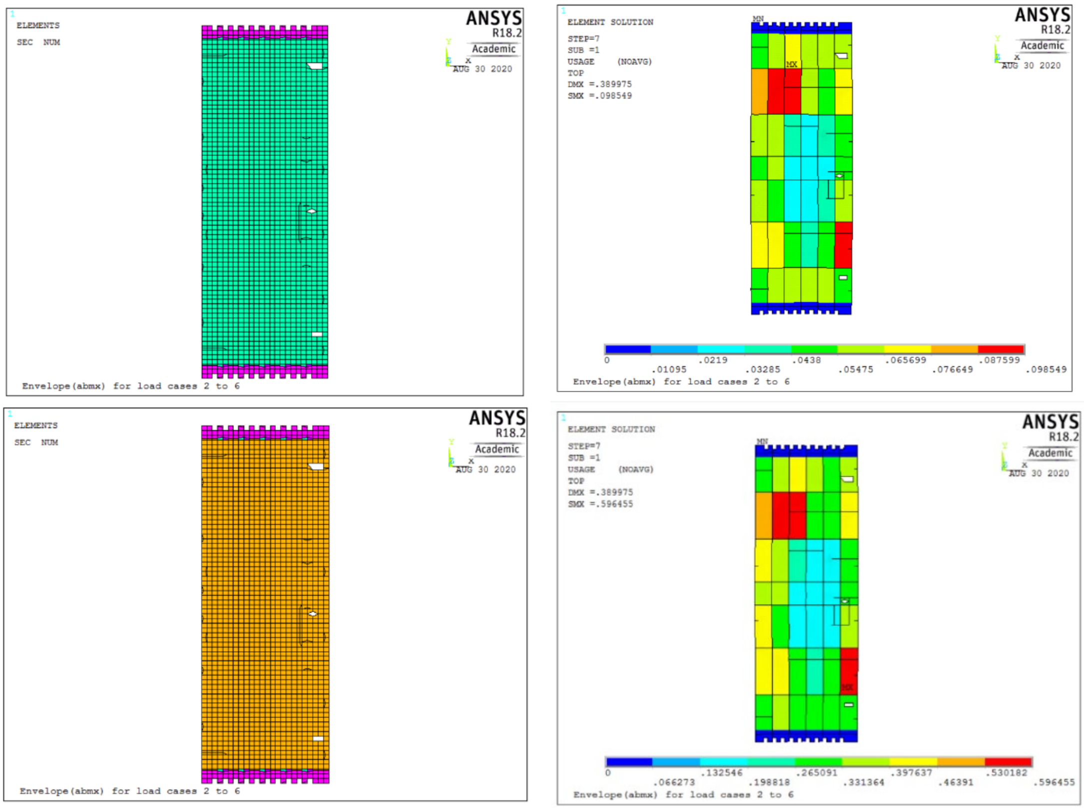


Figure 45. Usage Factor Plot for Deck at 7 m with plate thickness as 8 mm (above) and 4 mm (below)

It can be observed that the utilization factor pattern can be found to be similar in the two cases. However, the range of the utilization factor varies greatly – as the maximum utilization factor increases from 0.09 to 0.59 on reducing the plate thickness. Therefore, though the utilization criteria follow a similar pattern for the plates with differing thickness, the absolute value of the utilization factor increases sharply between the two cases.

5. Cumulative Panels Failed in the simulations

In the case of hogging, it can be seen that the highest increase in the percentage of panels failed from the first to the last iteration is at the Deck at 22.3 m for a utilization factor of 0.02 – where after 7% of the panels fail in the first iteration, 32% of the total panels fail when progressive failure stops.

Analysis	Part	Load Case	Utilization value	Percentage of Panels failed in the first iteration	Percentage of Total panels failed after complete simulation	Percentage Increase in Failure of Panels
1	Bulkhead	Hogging	1.50	12%	20%	67%
2	Deck at 22.35m	Sagging	0.08	32%	55%	71%
3	Full Model	Hogging	0.03	70%	100%	42%
4	Full Model	Sagging	0.03	70%	99%	41%
5	Full Model	Load Combination	0.10	37%	73%	98%
6	Full Model	Load Combination	0.06	50%	81%	60%
7	Deck at 22.35m	Hogging	0.02	7%	32%	367%
8	Deck at 2m	Hogging	0.05	58%	82%	42%
9	Full Model	Hogging	0.10	37%	51%	40%
10	Full Model	Hogging	0.06	50%	68%	35%
11	Deck at 7m	Hogging	0.06	31%	45%	46%
12	Deck at 22.35m	Sagging	0.15	14%	27%	100%

Table 5. Percentage failure results of the different analysis

In the case where the utilization factor of 1.5 is used to study the hogging load on the bulkhead, it can be seen that during the progressive collapse, the total number of panels that fail is quite low compared to the total number of panels. (The dotted red line indicated the total number of panels in the substructure).

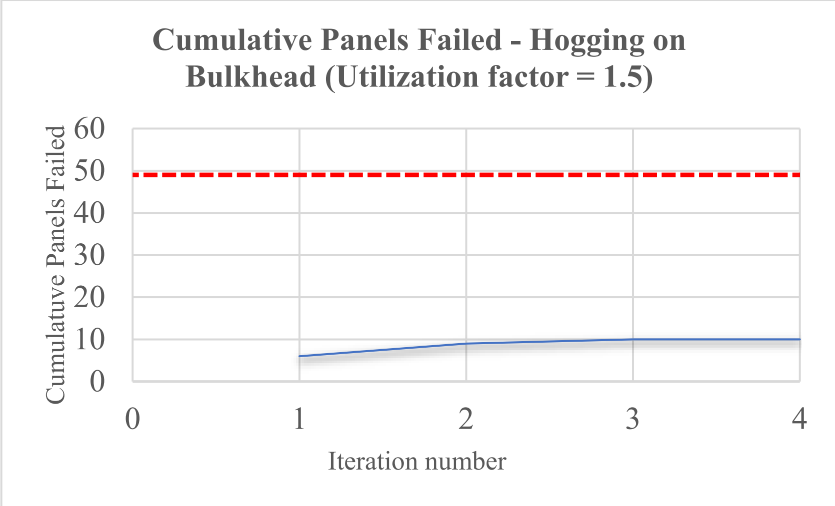


Figure 46. Cumulative Panels Failed - Hogging on Bulkhead (Utilization factor = 1.5)

In the case where the utilization factor selected is extremely low at 0.03, 70% of the panels of the full model fail at the first iteration itself – and progressively all panels fail. Therefore, this utilization factor is critical, or inversely the hogging load that corresponds to this utilization factor will cause all panels to fail.

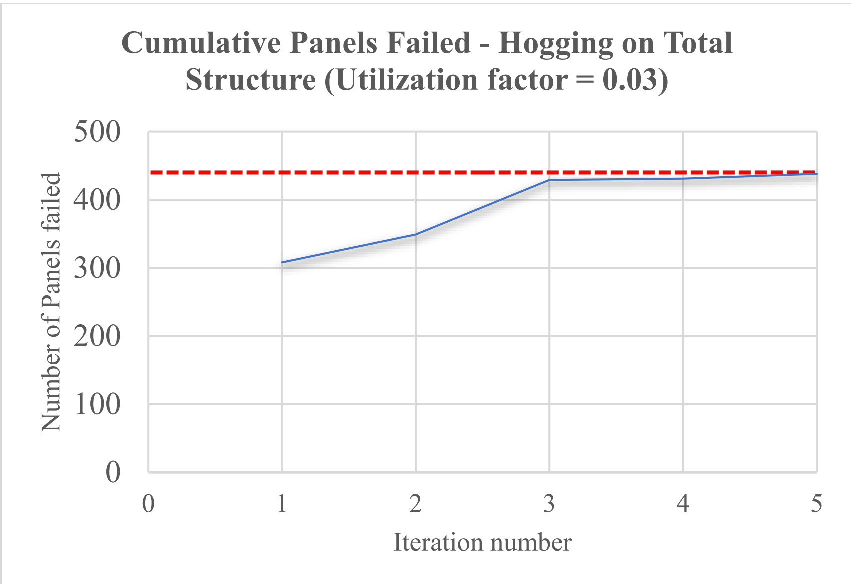


Figure 47. Cumulative Panels Failed - Hogging on Total Structure (Utilization factor = 0.0322)

In the case of sagging, it can be seen that the highest increase in the percentage of panels that failed from the first to the last iteration is at the Deck at 22.3 m for a utilization factor of

0.15 where there is a 100% increase in the number of panels failed from the first to the last iteration.

In the case for the sagging load acting on the Deck at 22.3m with a utilization factor of 0.08 – it can be seen that almost half the panels in the overall structure fail at the end of the simulation, and the structural integrity of the overall structure when over 50% of the panels fail needs to be verified further.

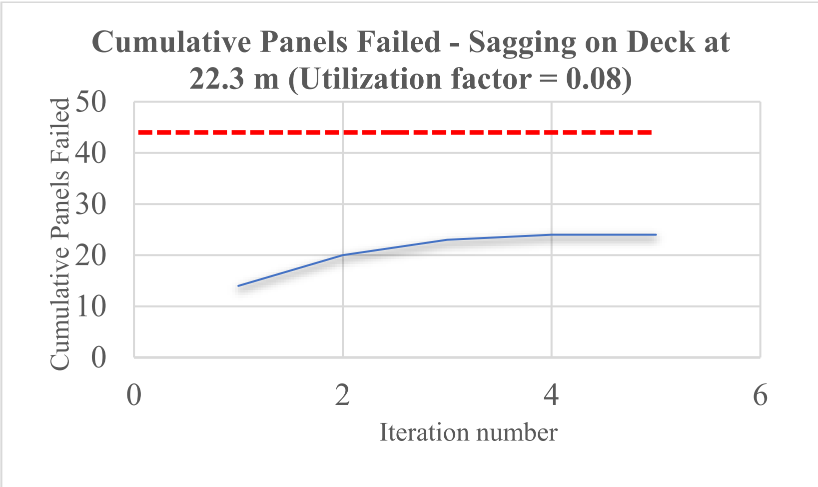


Figure 48. Cumulative Panels Failed - Sagging on Deck at 22.3 m (Utilization factor = 0.08)

For the load combination with alternating loads, it can be that the percentage increase in the number of panels failed between the first and last iteration is the highest at about 98%. This is an extreme case scenario which is rare – and it can be observed that the progressive application of the alternating loads causes an increasing number of panels to fail, as panels at the top and bottom undergo compression alternatingly.

In the case for the alternating load acting on the Main Model with a utilization factor of 0.06 – it can be seen that close to all the panels (81%) fail at the end of the simulations. However, the loads corresponding to these alternating loadings is extremely rare and as it is an extreme case scenario, it can be concluded that the structure will not experience such high loads during the vessel's expected operations, and designing the overall structure for this loading case be would be overdesign.

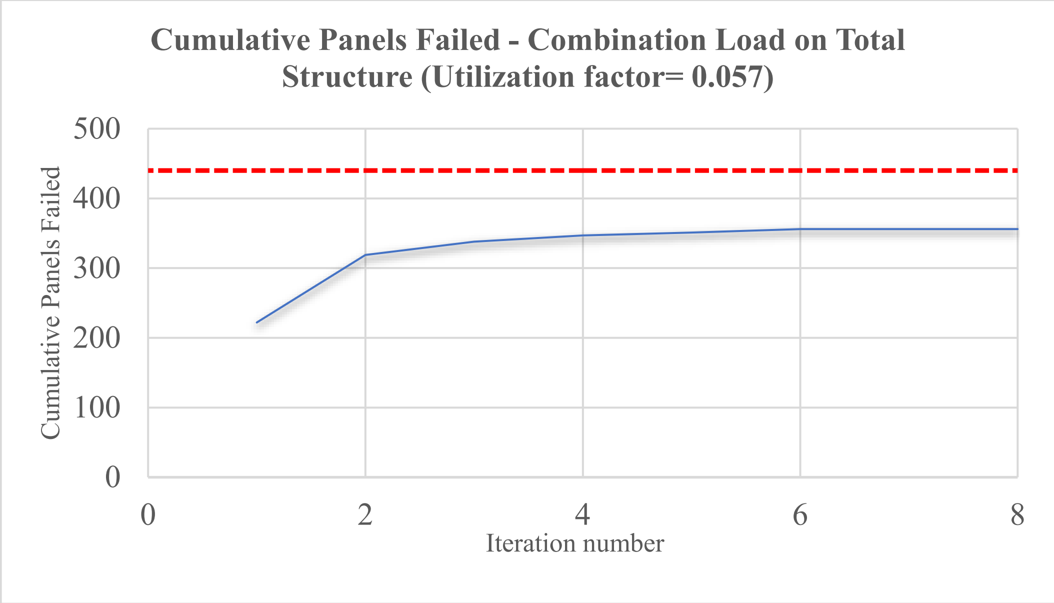


Figure 49. Cumulative Panels Failed - Combination Load on Total Structure (Utilization factor= 0.057)

5.9 Parameter Sensitivity of Buckling Resistance

In this section, the factors affecting the utilization criteria of the structure are studied – under different loading conditions, with an aim to improve the buckling resistance of the structure. The various loading conditions discussed in this section include longitudinal, transverse and shear loads, and the corresponding buckling stresses. Stiffened and unstiffened plates are studied, and the influence of the utilization criteria under the various loads for two different materials are presented here. (See Chapter 3. RULE-BASED BUCKLING STRENGTH ANALYSIS)

The materials:

Material 1: Steel grade A is the common tensile strength steel and has good toughness properties and high strength, good corrosion resistance, and good processing and welding properties. It is optimum for construction of ship hull structure (Grade A shipbuilding steel plate, n.d.).

Parameter	Value	Unit
Tensile, yield strength	235	MPa
Ultimate Tensile Strength	400-520	MPa

Material 1: Properties steel grade A(*UR W11 Normal and Higher Strength Hull Structural Steels - Rev.9 May 2017*, n.d.)

Material 2: Steel grade A36 is the high tensile steel used in marine structural engineering. The material is DNV approved and is suitable for construction of- cruise vessels, ferries, yachts, but also offshore oil drilling platforms or bulk carrier hull (DNV AH36 steel plate, n.d.).

Parameter	Value	Unit
Tensile, yield strength	355	MPa
Ultimate Tensile Strength	490-620	MPa

Material 2: Properties steel grade A36 (*UR W11 Normal and Higher Strength Hull Structural Steels - Rev.9 May 2017*, n.d.)

Buckling Resistance vs. Thickness of plate for different load cases

In assessing the buckling resistance, different load cases have been assumed, like uniform longitudinal compression, uniform transverse compression and shear stress acting on unstiffened and stiffened plates. For unstiffened plates, it is understandable that assessing longitudinal compression will give higher buckling resistance, whereas the lowest is achieved in transverse compression.

- **Unstiffened plate in Uniform longitudinal compression**

In the graph below, the buckling resistance in longitudinal compression is plotted against the plate thickness of the structure subjected to analysis. Therefore, considering the two types of material, steel grade A and steel grade A36, it can be observed that higher tensile steel can bear loads larger than the normal steel A, even from the start, where the thickness is low at 6mm. By increasing the thickness of the material and following the behaviour of the two materials, it is visible the fact that the steel grade A almost reaches the yield point by 18 mm thickness, whereas the higher tensile steel has better behaviour, detaching the curve from the initial path, and at the same thickness of 18 mm, the material reaches a strength of little above 300 N/mm^2.

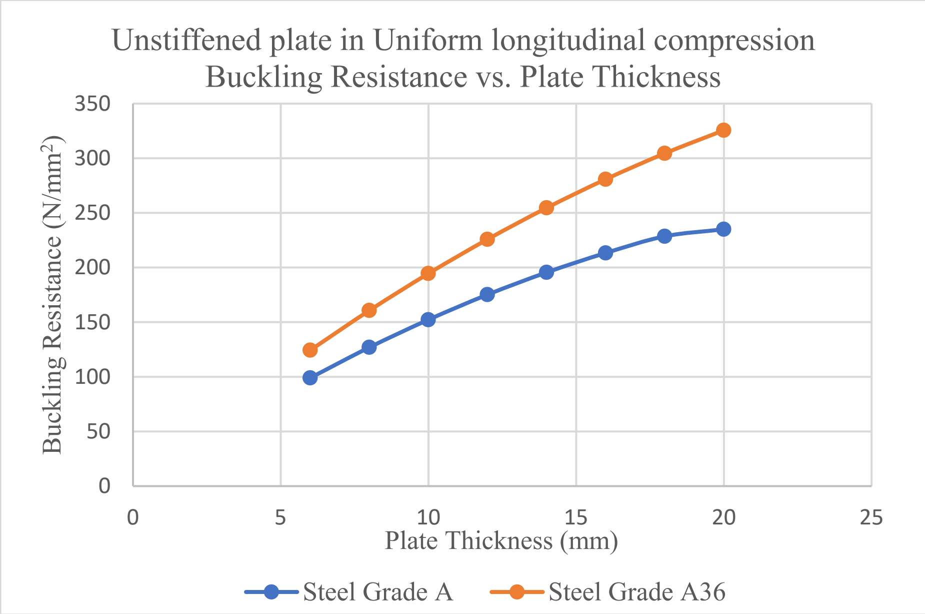


Figure 50. Unstiffened plate in Uniform longitudinal compression – Plot of Buckling Resistance vs. Plate Thickness

- **Unstiffened plate in Uniform transverse compression**

Similarly, to the behaviour at longitudinal strength, both materials provide a rising path proportionally to the thickness used in the structure, but in this case, the difference is not that large between the two. This validates the fact that buckling resistance is considerably lower when plates are subjected to transverse compression, giving no significant efficiency in using higher tensile steel if great results want to be achieved. By a difference of only almost 29 N/mm^2 at the considered peak, one might consider alternative measures of strengthening the structure against transverse loading, rather than utilising the higher steel grade A36.

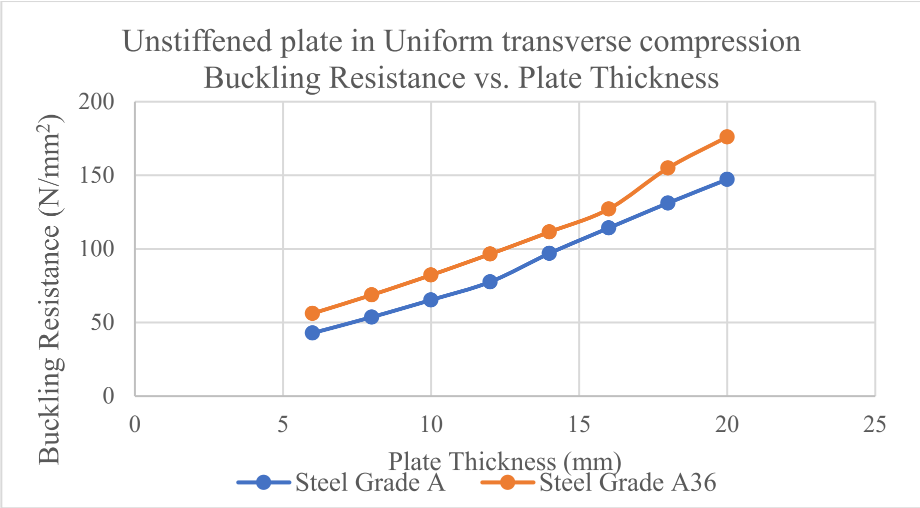


Figure 51. Unstiffened plate in Uniform transverse compression - Buckling Resistance vs. Plate Thickness

- **Unstiffened plate in Shear stress**

Regarding shear stresses, in the plot below it is displayed the behaviour of the two materials, steel grade A and steel grade A36, where both hit a sort of plateau, but at different thicknesses. In the first material case, the structure loses its rising buckling strength at a value of nearly 135 N/mm^2, whereas the second material keeps a better resistance profile by hitting a plateau at approximately 202 N/mm^2. The difference between the two values shows that material with better ultimate strength properties can face better the action of shear stresses on the structure.

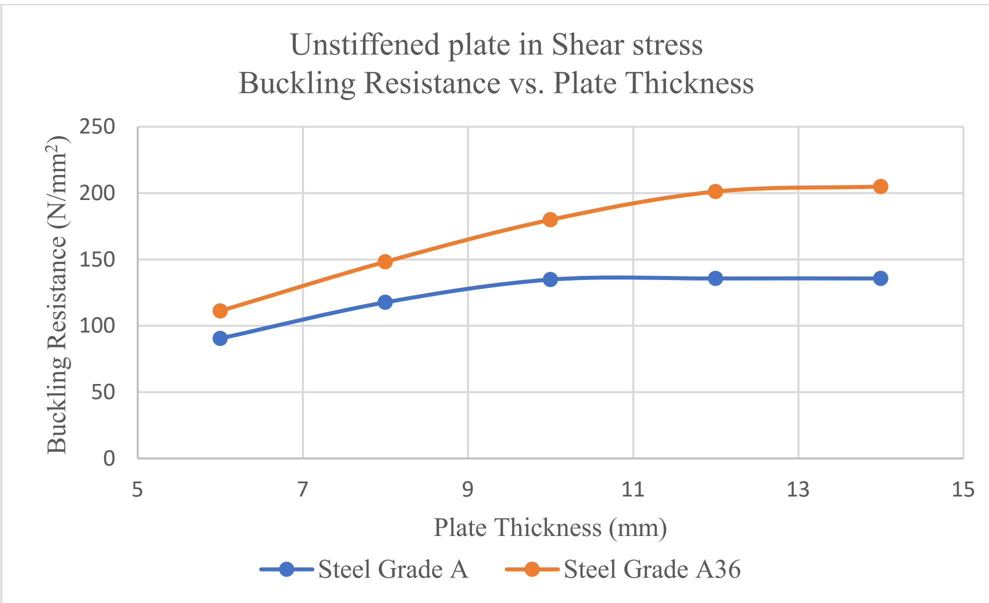


Figure 52. Unstiffened plate in Shear stress- Buckling Resistance vs. Plate Thickness

- **Stiffened Plate in Uniform longitudinal compression**

Compared with the anterior case, the unstiffened plate, when assessing a stiffened panel in the same loading case, under longitudinal compression, the structure does not have the same buckling resistance as in the previously mentioned situation. Here, the resistance has decreased with approximately 15 N/mm^2 for the steel grade A36 and by a little lower value for the normal steel grade A. From this graph can be understood the fact that nevertheless, the higher tensile material has a far better behaviour under compressive loads, the stiffened panels show more vulnerability when the load is longitudinally distributed.

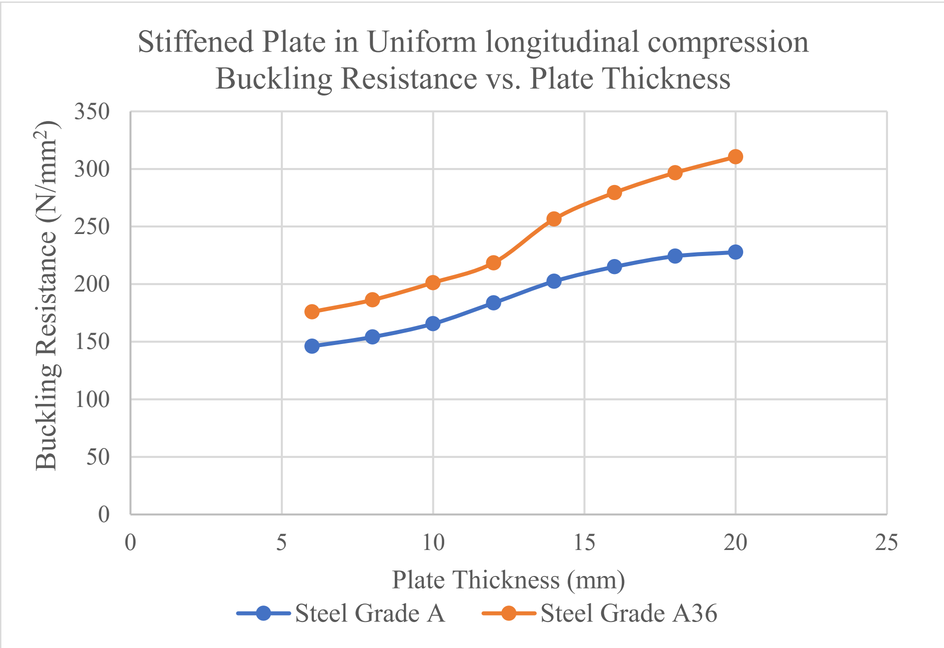


Figure 53. Stiffened Plate in Uniform longitudinal compression Buckling Resistance vs. Plate Thickness

- **Stiffened Plate in Uniform transverse compression**

In the transverse load distribution, the panels give a lot less strength resistance but compared to the unstiffened plates the values keep a sort of convergence, meaning that although the strength of the plate is higher, due to added stiffening, the strength resistance is maintained as in the previous situation, not improving the efficiency of adding more stiffening members.

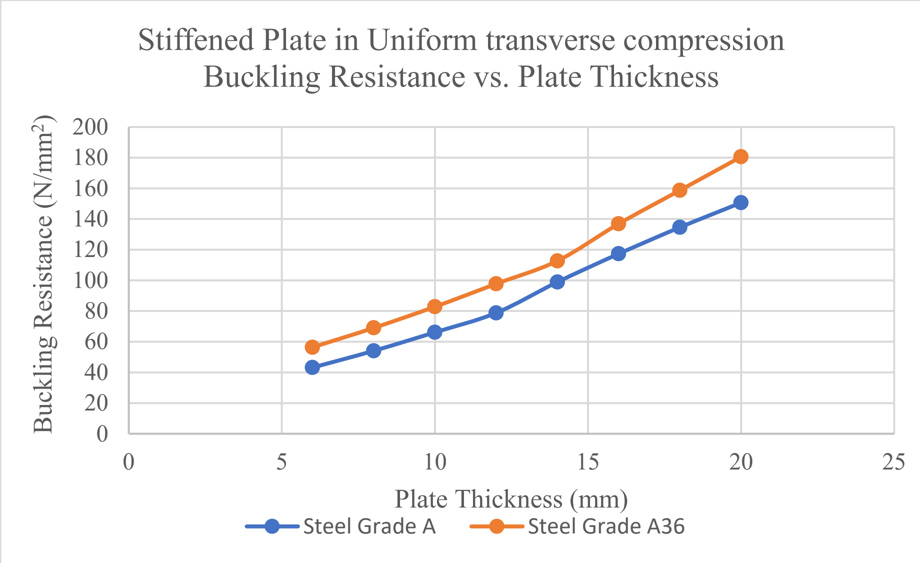


Figure 54. Stiffened Plate in Uniform transverse compression- Buckling Resistance vs. Plate Thickness

Usage Factor vs. Width of plate for different load cases

When assuming the width of the plate, all the material properties become fixed and the only parameter that can be changed for buckling strength assessment is the plate width. In the following section, the comparison between the material usage factor and the different plate widths is analysed.

Modulus of Elasticity	E	206000	N/mm^2
Plate Thickness	t	8	mm
Plate Width	s	1000	mm
Plate Length	l	2400	mm
Material Factor	γ	1.15	

Table 6. Material Properties

- **Plate with longitudinal compression**

Steel Grade A36 – in the graph below the usage factor is calculated considering the yield strength 355 N/mm^2. As can be observed in the legend, the blue curve is plotted by assessing the structure subjected to longitudinal compression at 150 N/mm^2, and the orange curve where for the same loading situation, the criteria is 100 N/mm^2.

By reading the plot, it is understood that the usage factor of the plate increases proportionally with the increase in plate width, respectively the buckling resistance decreases, by not having enough stiffening members to disperse the loads. By assessing the structure using material grade A36, it can be mentioned that the higher the strength properties of the material, the lower utilisation factor the plate material will have, resulting in higher buckling resistance for the steel grade A36.

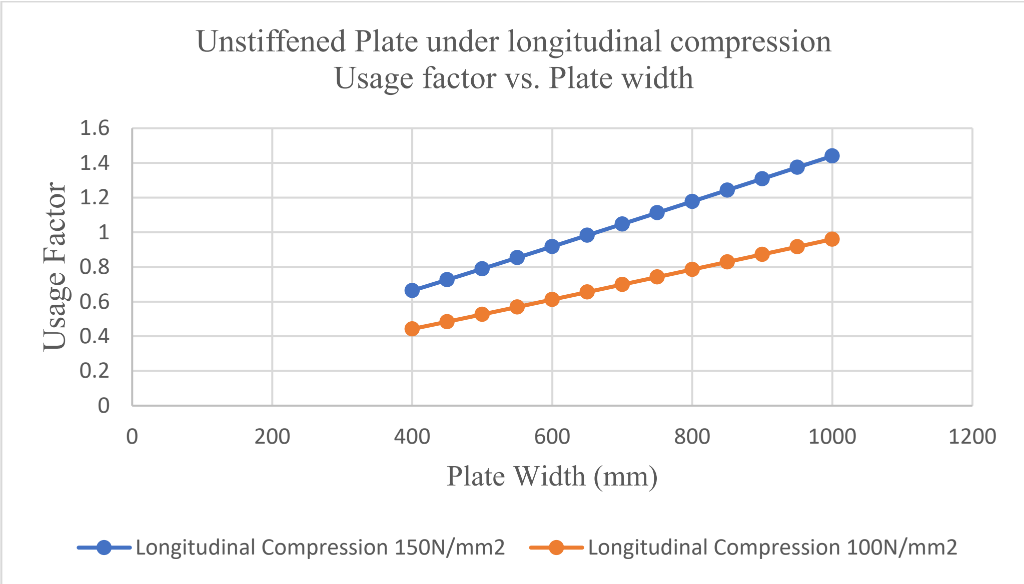


Figure 55. Steel Grade A36- Unstiffened Plate under longitudinal compression Usage factor vs. Plate width

Steel Grade A – the strength properties of the material are lower compared to the material considered above. The structural member is assessed considering the same longitudinal compression loads and the same criterion values. Unlike the steel grade A36, the normal steel gives a higher utilisation factor as its strength resistance it is much lower. Once again, similarly to the previous situation, the buckling resistance is decreasing proportionally to the increase in the width of the plate subjected to analysis.

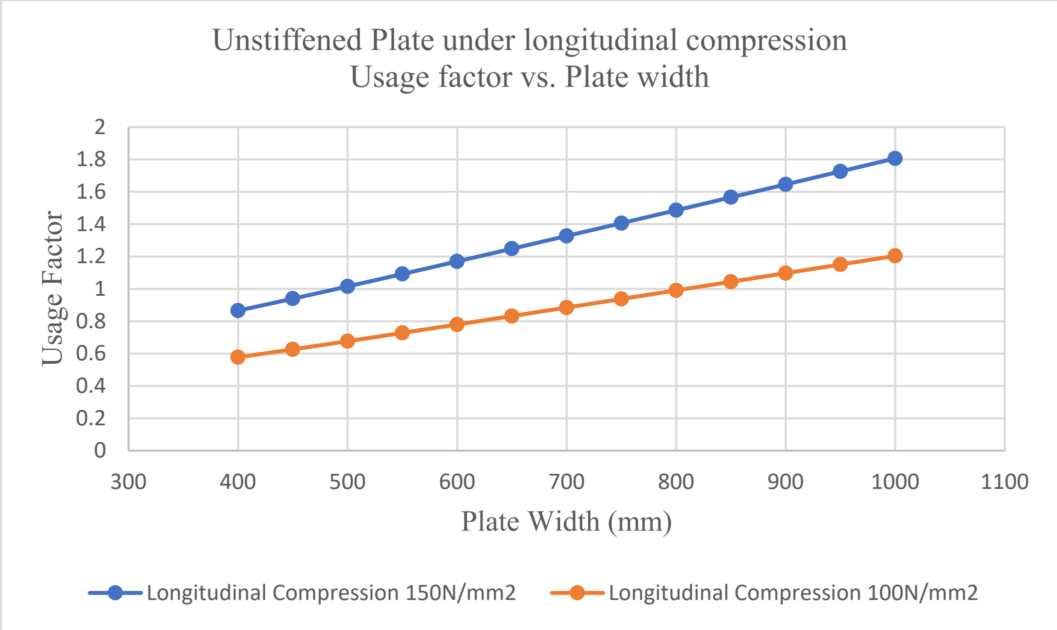


Figure 56. Steel Grade A- Unstiffened Plate under longitudinal compression
Usage factor vs. Plate width

In conclusion, it can be stated that the buckling resistance of a plate subjected to longitudinal compressive stress is higher than the one of a plate subjected to shear stress, that is in turn higher than the one of transverse stress.

6 CONCLUSION

From the various analysis carried out, it can be concluded that in most of the loading cases considered - the structure is multiple redundant as at the end of progressive failure analysis the loads are redistributed among the remaining panels and the structure does not fail. In the case of the loading combinations where alternatingly hogging and sagging loads are applied the entire structure does fail, however as these are extreme and rare scenarios, the structure can be considered to be multiple redundant for its everyday operations.

In analyses where greater than 70% of panels fail at the first iteration, the analysis ends with almost the whole structure failing. Therefore, it can be concluded that for the analysis where greater than 70% of the panels fail in the first iteration – the analysis can be stopped, and the structure needs to be redesigned.

The location of panels which are prone to buckling failure can be easily identified from the different analysis, and thereby when designing the vessel – care can be taken to ensure safety/emergency equipment are not kept in the vicinity of these panels.

The different parameters affecting the buckling resistance value and thereby the usage factor are also discussed, and it can be seen that the longitudinal compressive stress of a plate is of more importance that the plate subjected to shear stress, which in turn is greater than the transverse compressive stresses. Also stiffened panels with large stiffeners provide better buckling resistance and increasing the number of stiffeners improves the buckling resistance of the structure as well. However, the weight of the structure needs to be kept in mind while carrying out such design considerations.

Therefore, the progressive collapse analysis of the structure proves to be beneficial in structural design. However, limitations exist to this procedure – and the analysis of this progressive collapse failure is a vessel-specific analysis. Repeated analysis of different vessels and different types of vessels is required before universal guidelines applicable to all vessels can be developed.

7 FURTHER WORK

The following can be carried out in order to improve the various methods used in this thesis.

- **Non-linear finite element analysis**

Non-linear finite element analysis can be used to improve and compare the accuracy of the results of the analysis.

- **Removal of girder and other elements**

Panels are considered only between the transverse and longitudinal girders. Adding girders to the analysis will give more realistic results.

- **Redistribution of removed loads**

When a panel is removed at the end of each iteration, a part of the load also gets removed. This is due to constraints using ANSYS. Every time a panel is removed, the loads on the adjacent panels can be redistributed in order to get more precise results.

- **Use of flexible macros**

Macros which can identify panels which are not bounded stiffeners on all side, curved panels and panels with both transverse and longitudinal stiffeners thereby improving the detail of the model can also be considered.

- **Yield and Deflection checking**

  Yield and Deflection checking of the panel can be used to analyse different failure scenarios.

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Midhun Kanadan

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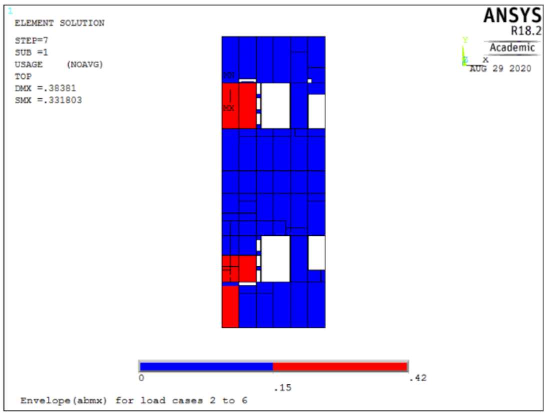
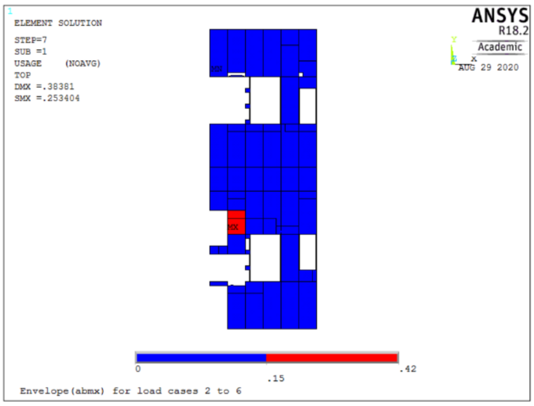
A. APPENDIX 1 - PROGRESSIVE FAILURE OF PANELS AND USAGE PLOTS FOR DIFFERENT ANALYSIS

Analysis 9 - Sagging Load on the Deck at 22.35m

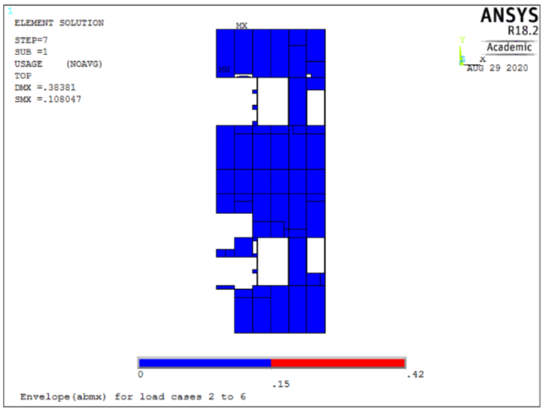


Usage Plot: First Iteration

Usage Plot: Second Iteration

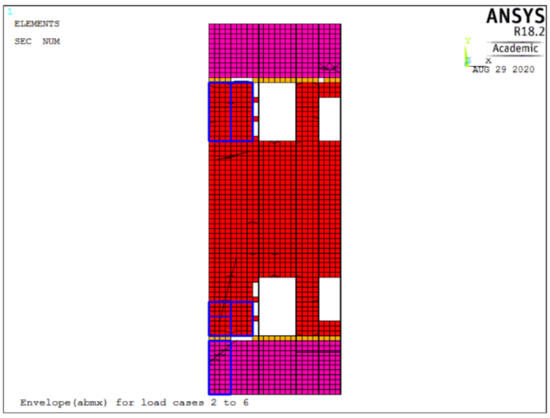


Usage Plot: Third Iteration

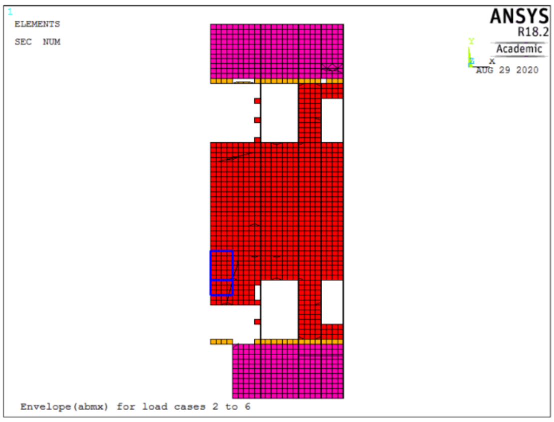


Usage Plot: Fourth Iteration

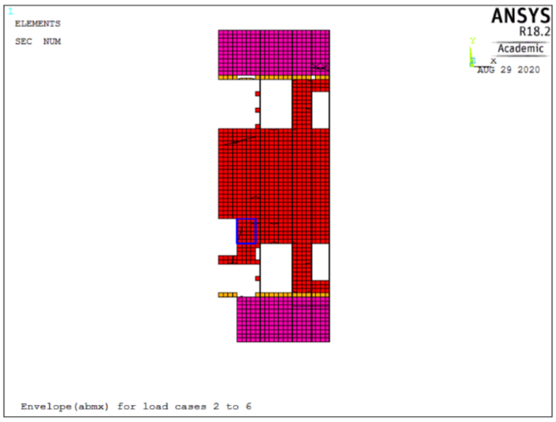
Figure 57. Usage Plot for a Deck at 22.35 of the Main Model under sagging load, utilization criteria 0.15



First Iteration



Second Iteration

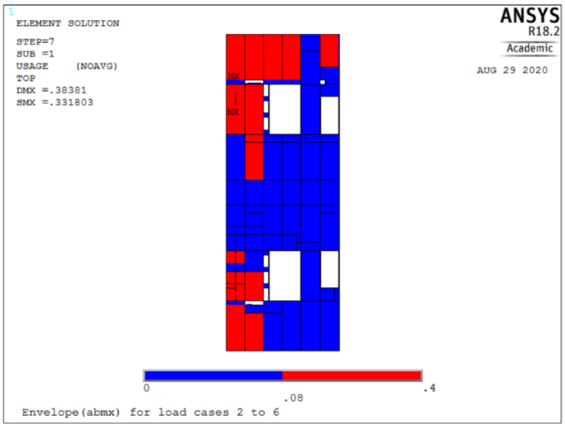


Third Iteration

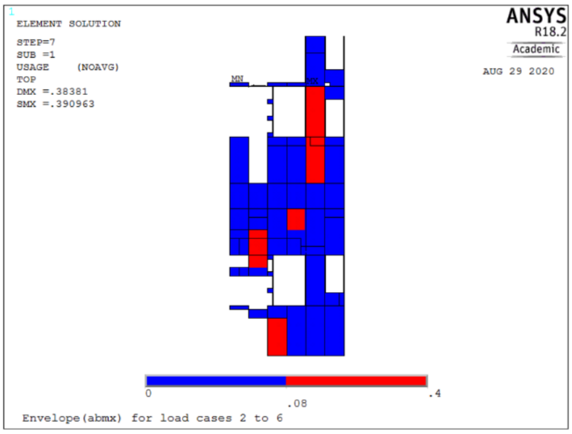
Critical Path

Figure 58. Progressive Panel Failure for a Deck at 22.35 of the Main Model under sagging load, utilization criteria 0.15

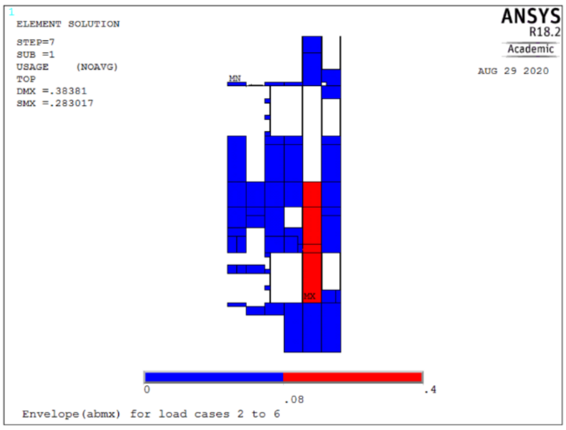
Analysis 10 - Sagging Load on the Deck at 22.35m



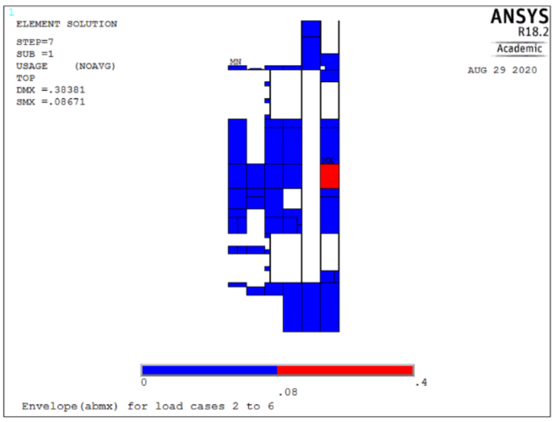
Usage Plot: First Iteration



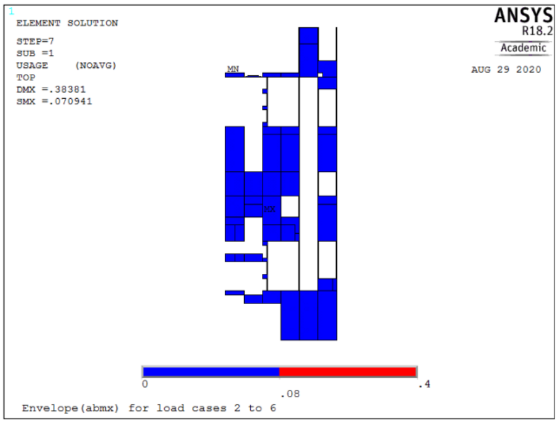
Usage Plot: Second Iteration



Usage Plot: Third Iteration

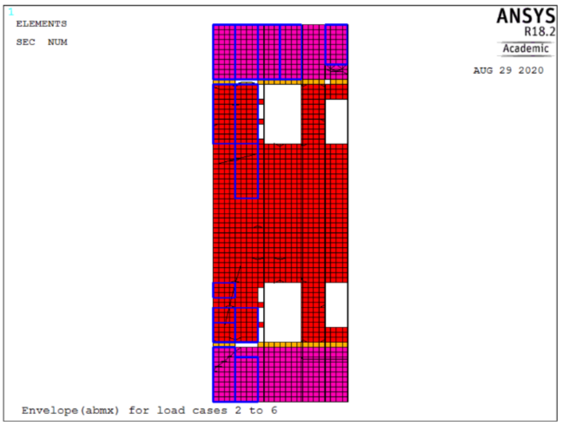


Usage Plot: Fourth Iteration

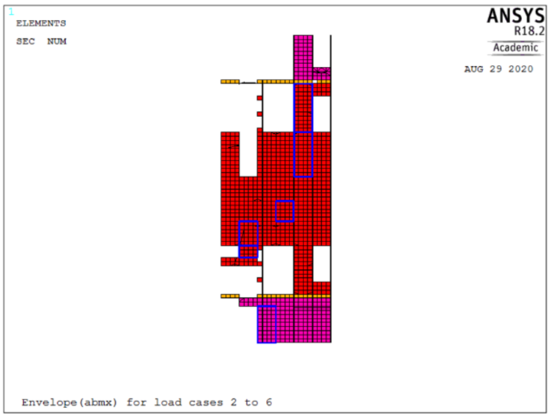


Usage Plot: Fifth Iteration

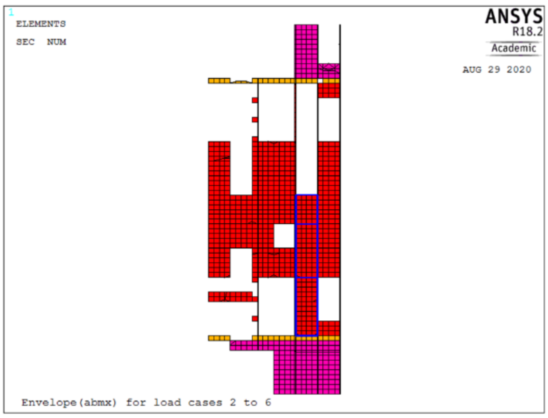
Figure 59. Usage Plot for a Deck at 22.35 of the Main Model under sagging load, utilization criteria 0.08



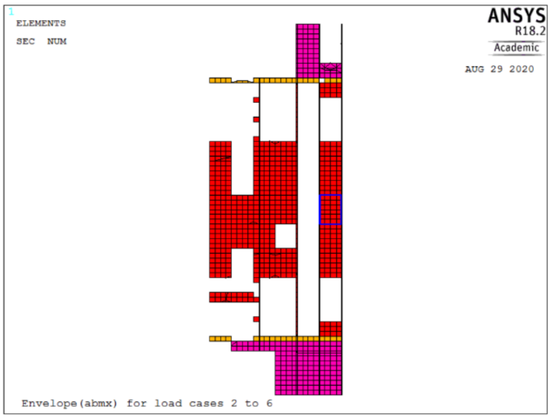
First Iteration



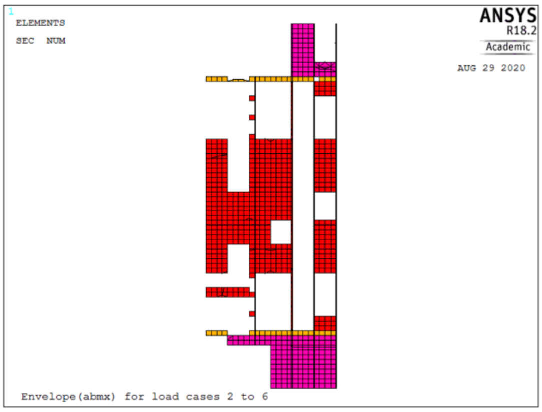
Second Iteration



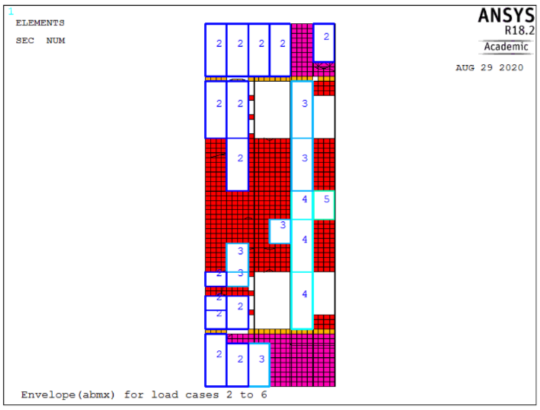
Third Iteration



Fourth Iteration



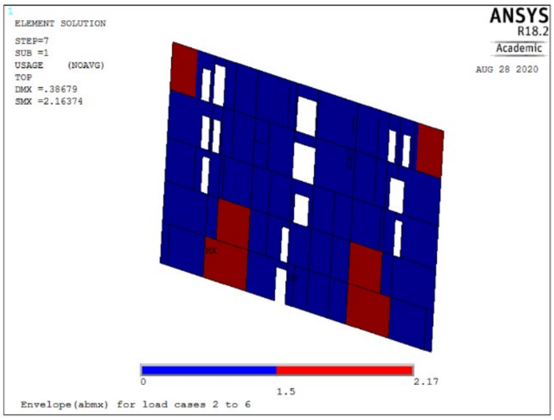
Fifth Iteration



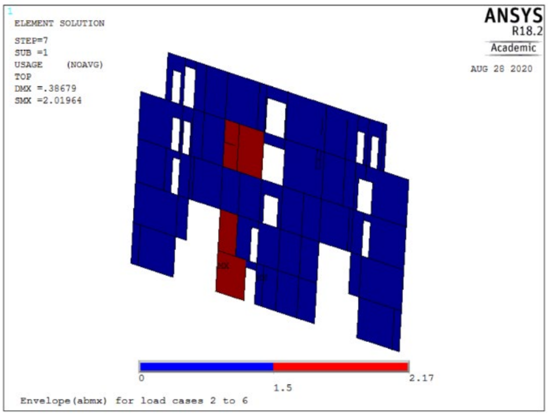
Critical Path

Figure 60. Progressive Panel Failure for a Deck at 22.35 of the Main Model under sagging load, utilization criteria 0.08

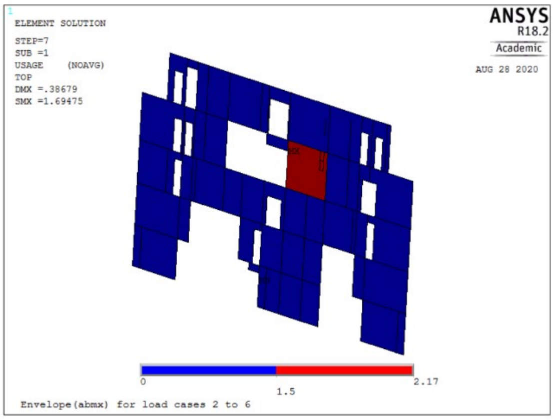
Analysis 11 - Hogging Load on the Bulkhead at 154.8m



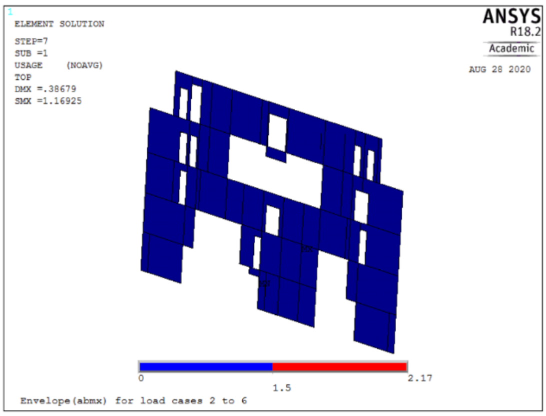
Usage Plot: First Iteration



Usage Plot: Second Iteration

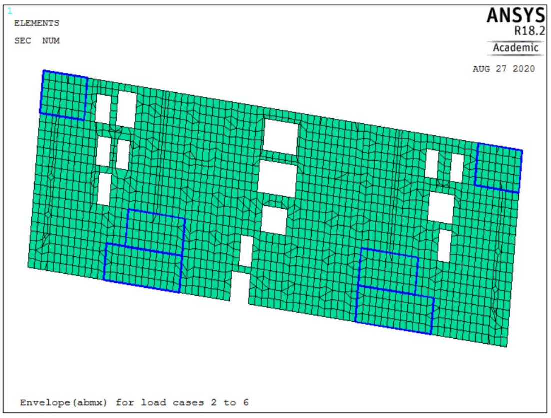


Usage Plot: Third Iteration

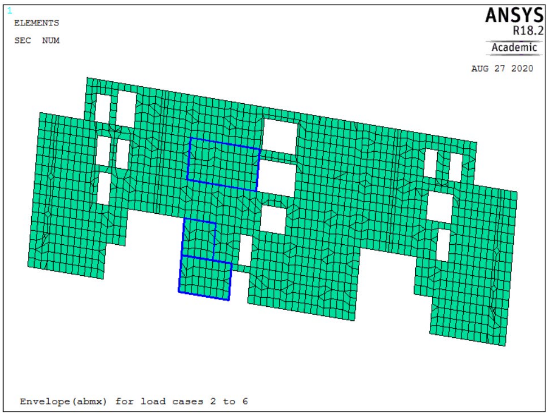


Usage Plot: Fourth Iteration

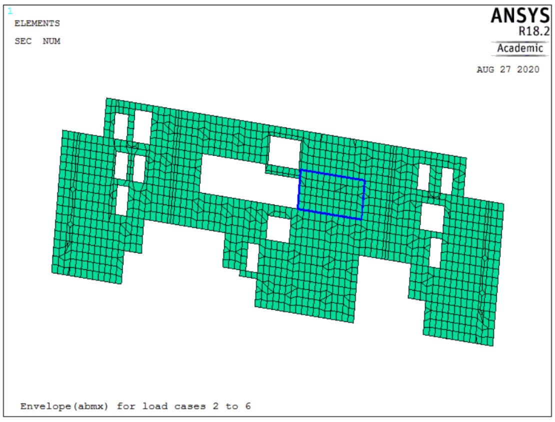
Figure 61. Usage Plot for a Bulkhead at 154.8m of the Main Model under hogging load, utilization criteria 1.5



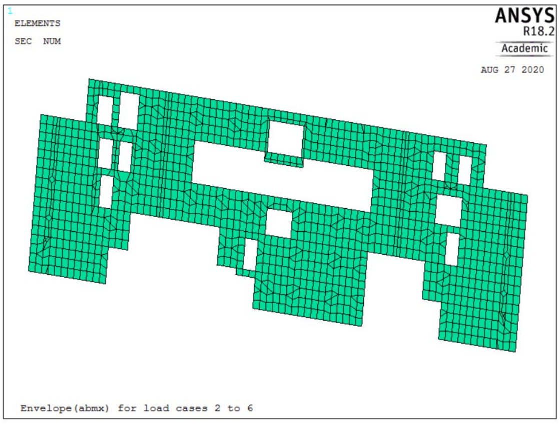
First Iteration



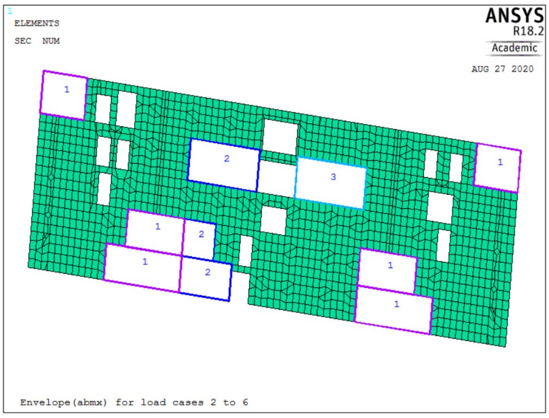
Second Iteration



Third Iteration



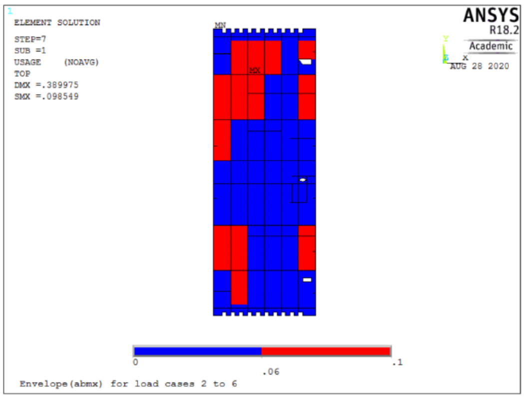
Fourth Iteration



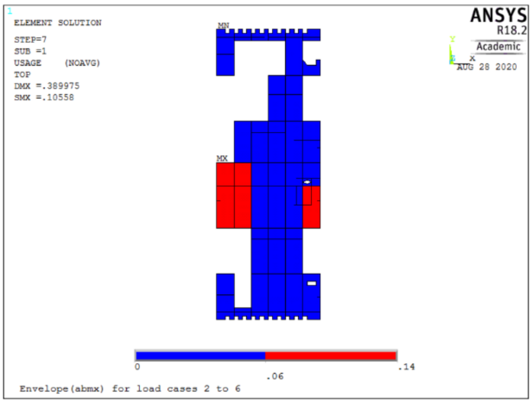
Critical Path

Figure 62. Progressive Panel Failure for a Bulkhead at 154.8m of the Main Model under hogging load, utilization criteria 1.5

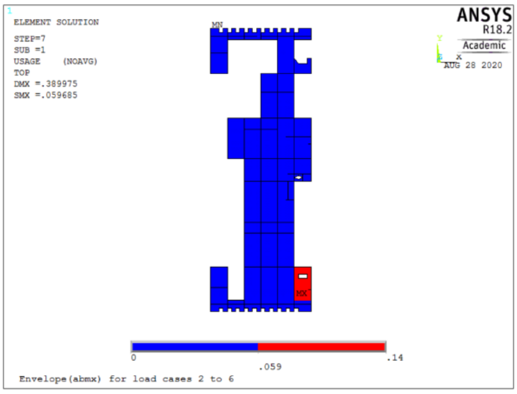
Analysis 12 - Hogging Load on the Deck at 7m



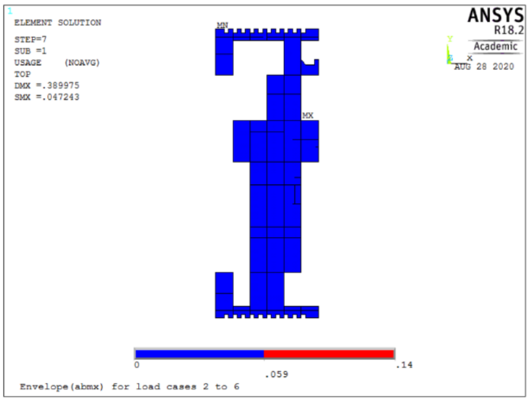
Usage Plot: First Iteration



Usage Plot: Second Iteration

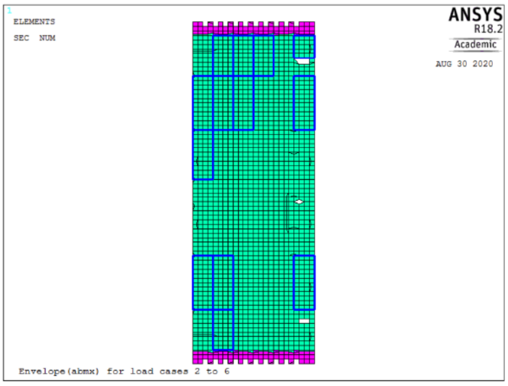


Usage Plot: Third Iteration

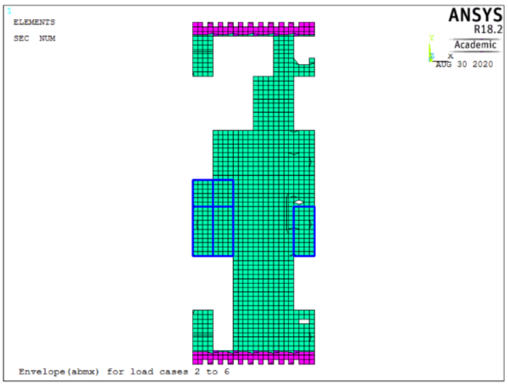


Usage Plot: Fourth Iteration

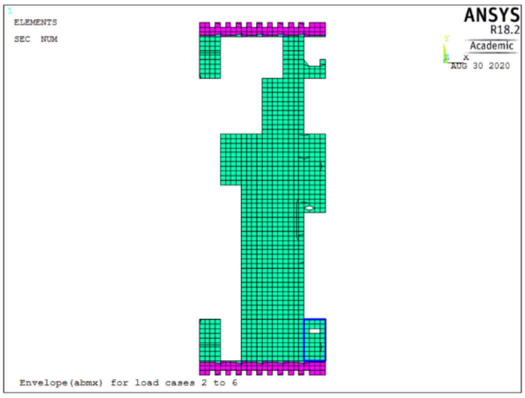
Figure 63. Usage Plot for a Deck at 7m of the Main Model under hogging load, utilization criteria 0.06



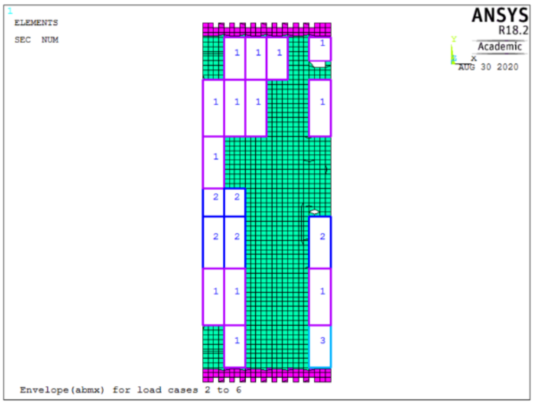
First Iteration



Second Iteration



Third Iteration



Critical Path

Figure 64. Progressive Panel Failure for a Deck at 7m of the Main Model under hogging load, utilization criteria 0.06