CHAPTER ONE

Introduction

1.1 Background of the Study
The volume of greenhouse gases generated and emitted worldwide is believed to be on the increase due to both natural (volcanic eruption, plate tectonic) and anthropogenic reasons. The IPCC (2007) assessment summary report for policymakers reported that ‘global greenhouse gas emission due to human activities have grown since pre-industrial times, with an increase of 70% between 1970 and 2004’. Though in Europe in the last few decades greenhouse gas emissions is perceived to have been on the decrease due to concerted efforts in cutting down emissions in line with international commitment (European Environment Agency, 2015) but, the case in other areas such as China, India and other developing and developed countries may not be the same. It is also argued that, unlike human migration where border control may suffice in cutting down entry, global greenhouse gas emission is not a respecter of border control. Therefore, the net resultant effect may still be there, the reason why some in the industry think achieving the 2°C target for the low emission scenario is probably unrealistic giving noncompliance by some nations. The Conference of Parties (COP 21) in Paris 2015 gives strength to this argument as well the IPCC (2007) assessment summary report reporting that ‘There is high agreement and much evidence that with current climate change mitigation policies and related sustainable development practices, global greenhouse gas emissions will continue to grow over the next few decades’. Whatever the argument, global emissions of greenhouse gases leads to atmospheric pollution which does not only affect the natural environment but can also accelerate the deterioration and corrosion of engineering materials and infrastructural systems (Moncmanová, 2007). It is a general scientific consensus that is being supported by several authors and multilevel research that the major source of these
greenhouse gas emissions is anthropogenic i.e. from human activities due to rapid industrial and agricultural activities that release carbon dioxide (CO₂), methane (CH₄), chlorofluorocarbon (CFC) and nitrogen dioxide (N₂O) (MetOffice, 2011). All of these gases contribute to the greenhouse effect at a different warming potency (Webster et al., 2011). These greenhouse gases create a canopy in the atmosphere and trap the solar radiation reflected back from the Earth’s surface leading to atmospheric and climatic changes (Sharma, 2006). Figure 1.1 shows how some of the infrared radiation passes through the atmosphere while most is absorbed and re-emitted in all directions by the greenhouse gases and the clouds.

Climate change is a consequence of global warming i.e. a result of the huge increase in greenhouse gases in the atmosphere due to deforestation and burning of fossil fuel (Maslin, 2013). The latter is measured in terms of the aforementioned gases’ ‘greenhouse warming potency’ (GWP) i.e. the measure of any of these gases’ contribution to global warming as compared to the same mass of carbon dioxide contribution (Webster et al., 2011). It is reckoned that CO₂ is not the most potent greenhouse gas but, it is rather the most abundant, and therefore it has the largest cumulative effect. It is this effect that results to long-term climate change and most often extreme weather events and variability.

Figure 1.1 Greenhouse effect (IPCC working group 1 support unit)

Throughout the world, global climate change impact and environmental deterioration of material is being researched with a view to understanding and identifying the current status of scientific knowledge concerning climate change impacts, vulnerability, adaptation and mitigation. The key figure in promoting this research is the Intergovernmental Panel on
Climate Change (IPCC), based in Switzerland. Their work is informed by modelling carried out by the Hadley Center of the UK Metoffice, the Max Planck Institute Germany and the National Oceanic and Atmospheric Administration (NOAA) in Washington DC, USA. The IPCC (2013) report establishes that climate change exists with a 95% certainty that humans are responsible.

Environmental deterioration of materials is a complex interaction between the effects of climatic and meteorological variables with chemical processes; in some cases, biological processes can become important as well. Generally, environmental deterioration mechanism of materials and corrosion of metals include erosion, volume change of material, dissolution of material and its associated chemical changes and biological process. The principal environmental factors causing deterioration apart from decay from catastrophic one-off events are moisture, temperature, precipitation, solar radiation, air movement and pressure, chemical and biochemical attack, intrusion by micro and macro-organism. These factors have an effect on damaging processes like mechanical stress, desiccation, surface scaling, attrition and cracking. When buildings and constructional material degrade, it leads to reduction of the service life of such engineering systems with both economic and structural integrity consequences. Webster et al. (2011) suggested that structural engineers have a responsibility of finding ways to reduce climate change impacts on structures. For example, when high storm intensity becomes more intense and frequent, current design criteria may become obsolete for design of new structures. Furthermore, for existing structures, their inherent (inert) capabilities may degrade over time and this will pose safety and integrity challenges for these (Hirsch, 2010). SCOSS, in its 13th Report as cited in Nethercot (2003) expressed concerns such as: “The naturally occurring environmental hazards that threaten the structural safety of a building, bridge and any other civil engineering structure arise primarily from the climate and sometimes naturally occurring chemicals or earthquake phenomena at the location of the structure.” Furthermore, they outlined the main climatic and meteorological hazards challenging the structural engineer as being extreme wind and snow, rain and ice, temperature, flooding, scouring, settlement and ground instability. Nethercot citing SCOSS reported that the risks associated with these climatic hazards are usually controlled through prediction of extreme climatic events based on historical data. This is because generally in structural engineering future risk is assumed to be same as historical risk.
However, the use of past historical data to predict future risk in the face of changing climate might not be sufficient because of the randomness involved (Li et al., 2011).

Structural safety, health and integrity are important aspects of structural design for the overall satisfactory structural performance of the structure during its design life. Structural integrity is the quality of a structure being whole and complete or the state of being unimpaired as a result of soundness of design and construction. It expresses whether the structure is fit to withstand the service conditions safely and reliably throughout its predicted lifetime (Motarjemi and Shirzadi, 2006). As a subject, Steve Roberts as cited in (James, 2012) defines it as the science and technology of the margin between safety and disaster. Structural integrity ensures that the structure stands up better to detrimental effects and has high level functionality and safety. Structural integrity integrates the concepts of stress analysis, material behaviour and failure mechanics in design process.

The British Standards as well as the Eurocodes design philosophy is that the strength of the material used for the structure should be higher than the maximum applied load. This aims at achieving an acceptable probability that the structure will perform satisfactorily during its intended life span. Secondly, the structure should be able to, with a reasonable degree of safety, sustain all the loads and deformation of normal construction and use at the same time have adequate durability and resistance to the effects of misuse and fire (Motarjemi and Shirzadi, 2006). BS EN 197: Part 1 (2000) provides that a structure should be durable i.e. the structure should not deteriorate unduly under the action of environmental loads (actions) over the design life span. BS 8110: Part 1 (1997) adopts the limit state philosophy which requires that a structure designed should not reach its limit state i.e. should not become unfit for use either by collapse, overturning, buckling and fatigue, meaning the structure should remain in a condition where it is fulfilling the relevant design criteria (EC3, 2001; McCormac, 2008).

Durability is considered the ability of a structure, structural component or material to endure certain forces or processes in its environment of existence. Steele et al. (2003a) views durability as a function of the material, construction quality and maintenance. Durability is of concern to both the structural engineer and the infrastructural manager in that a designed structure should not deteriorate significantly with time because of the implication of deterioration on the serviceability of the structure with potential economic impact on the
client. Steele et al. (2003a) further assert that a reduction in durability will speed up deterioration and the need for maintenance and eventually replacement, which will increase the net environmental impact of the structure’s life cycle. Examples of some of these engineering structures that will be required to be durable in their environment of existence are steel and concrete bridges, buildings, ships and offshore structures, aircrafts, motor vehicles and telecommunication masts. In particular, highway and railway bridges play a significant role in the transportation sector throughout the world. Bridges can also serve as landmark iconic structures for example, the New York Brooklyn Bridge and the Forth railroad Bridge in Scotland.

1.2 Problem Statement
Bell (2007) reported the outcome of a demographic survey carried out for the Sustainable Bridge project indicating that, more than 35% of the over 300,000 European railway bridges are over 100 years old, with metallic bridges constituting over one third of the population. The same report revealed that the main asset management priorities have to do with the improvement of the assessment process and performance prediction (Kallias et al., 2016). It should be noted that the objective of bridge management is to allocate and use the limited resources available to balance lifetime reliability and life-cycle cost in an optimal manner (Frangopol et al., 2001). Again, prioritisation of limited funds to cater for the urgent needs in bridge maintenance, retrofit and rehabilitation activities is a huge challenge experienced by bridge management authorities worldwide (Das, 1999). For example, in the UK the road transport network is recognised as a key enabler of the economy (Eddington, 2006). However, it is also identified as the UK’s most expensive asset (DfT, 2005). The road transport network is reported to have 4,300 miles of strategic network under the care of the Highways Agency (now Highways England) valued at £88bn (Highways Agency, 2009), with another 183,300 miles of local roads under the management care of 152 highway authorities (DfT, 2014). Currently, this network is being used by 34 million licensed vehicles and the number is projected to have been on the increase every year since 1950 (DfT, 2013). Furthermore, the increase is being viewed to outgrow capacity leading to congestion problems being experienced. This congestion is said to have a resultant cost effect worth between £7-8 billion per annum on the UK’s economy (Eddington, 2006).
It should be noted that bridges are an essential part of the transport infrastructure whether road or railway. A considerable number of these bridges are metallic and aging; in many cases such metallic bridges are exceeding 100 years of age having suffered deterioration from environmental attack such as atmospheric corrosion (Fom et al., 2015). With the general acceptance that the Earth climate is changing, this is a significant factor to be considered that may likely affect the integrity of such valuable infrastructural assets that are almost running at full capacity more so a large proportion of these bridges lie on heavily utilised networks (Kallias et al., 2016). Edwards (2002) and Thornes (1992) have both reported that meteorological hazards are frequently an issue causing disruption and accidents. This report is also being corroborated by the IPCC (2014) adding that a good number of the risks suffered by these bridges are likely to worsen under the impact of climate change which may potentially lead to significant effects across all infrastructural systems. Defra (2012) has also reported that the transport sector is likely to be the most susceptible to the effects of climate change. Hall (2010) views climate change challenge as the most serious issue for the transport sector. Therefore, achieving resilience on the transport network is a very crucial matter in order to enable people, goods and services to move unhindered (Chapman, 2015). The Cabinet Office (2011) views infrastructural resilience from the perspective of resistance (protection), reliability, redundancy (spare capacity) and recovery.

Climate change effects can be viewed from two perspectives; increasing magnitude and frequency of extreme weather and climate events as well as long-term slow changes in average climatic, meteorological and atmospheric conditions. For example, extreme weather events define severe or unseasonal weather that is significantly different from the average or usual weather pattern such as flash floods, heat and cold waves, droughts, intense rainfall and hurricanes (IPCC SREX, 2012). According to the (NCDC, 2012) report provided by the National Oceanic and Atmospheric Administration Data Center, the annual aggregated loss due to extreme weather and climate events since 1980 for disaster events with nominal losses not less than $1 billion accounts for roughly 80% of the total cost ($880B out of $1,100B) for the period 1980-2011. Furthermore, these estimates only reflect the weather-related disaster losses in the US due to direct damage to assets alone not accounting for indirect losses to human lives, cultural heritage and ecosystem services because these are not easily monetised.

Climate change from fossil fuel combustion is viewed the most challenging environmental issue the world is facing today because of CO₂ emission which is at the heart of modern
standard of living (Archer, 2007). The agricultural revolution, which supports the over 7 billion human population on Earth today is supported by a heavy industrial production of fertilisers (Archer, 2007). Movement of goods and services across geographical space is been supported by fossil fuel. The energy that drives the global economy, a chunk of it is from fossil fuel. So, it is a challenge to stop CO₂ emission and countries and companies that emits lots of CO₂ have a very strong inclination to continue (Archer, 2007). It should be noted that, the climate-forcing agent at the heart of global warming is the greenhouse effect from the rising of CO₂ concentration in the atmosphere (Archer, 2007). This CO₂ gas makes it difficult for energy leaving the Earth to escape to space thereby influencing the Earth energy flow.

Structural deterioration of built infrastructural assets due to the combined effect of climatic parameters (temperature, relative humidity, precipitation, wind) and pollutants (SO₂, O₃, Cl⁻, NO₂) in a changing environmental condition can now be linked together by state-of-the-art deterioration models called dose-response functions (Kumar and Imam, 2013). Built infrastructure refer to both buildings and transport infrastructure (roads, railway tracks, bridges, earthworks, seaports, airports) that exist in this changing environment. The impact of this polluted environment can be seen in terms of material deterioration and blackening of buildings and such impacts are chronic and will generally take place over a long period of time (Kumar and Imam, 2013). Generally, quantification of material loss is done through the use of the dose-response functions which relate climate parameters with the atmospheric concentration of pollutants (Kucera and Fitz, 1995). Previous research have shown how to estimate material loss due to varying changes in the climatic parameters and ambient pollutants concentration. For example, corrosion assessment of heritage buildings in Italy was carried out by Screpanti and De Marco (2009). They discovered that for limestone and copper the corrosion rates were well above the tolerable levels, therefore suggested the need to reduce ambient concentration of Ozone (O₃) in that region. Air pollution engendered atmospheric corrosion of metals in Europe as well as the surface damage to modern concrete buildings was considered by Tidblad (2012) and Ozga et al. (2011) respectively. Clearly, from multilevel research outcomes such as those of Brimblecombe and Grossi (2007), Corvo et al. (2010), Haines et al. (2006), Sabbioni et al. (2006) and Varotsos et al. (2009) valid concerns have been raised over damages done to structural assets and their material as a result of changing environmental conditions. It should be noted that both chemical pathway (pollutants) and changing environmental conditions are equally important when it comes to
safety and economy of the transport infrastructure (Kumar and Imam, 2013). The huge value of these assets is indicative of the degree of risk that may be involved in terms of economic losses due to the effect of both climate change and extreme weather conditions. It should be noted that the highway and railway networks in the UK alone have assets in excess of £88 and £35 billion respectively (Highways Agency, 2009; Network Rail, 2009).

Pollutant-induced deterioration is realisable over a long-term and the process may increase the rate of damage to construction material such as steel, concrete and timber (Kumar and Imam, 2013). However, extreme events have a huge impact in the short term affecting road and rail networks (Booij, 2005; Nicholls, 2004; UNEP, 2007) with a colossal economic loss (Larsen et al. 2008). It should also be noted that there exist interdependencies between transportation infrastructure and utility services infrastructure such as electricity, telephone, gas networks and water pipelines. Therefore, the economic cost of transport assets and network failures may extend far beyond the boundaries of transportation systems alone but also affecting other critical infrastructure (ICE, 2009). Bridge failures in Cumbria, UK in 2009 due to extreme flooding demonstrates this interdependency as other critical infrastructure were affected and not only the transport connectivity (Stimpson, 2009). Considering these impacts underscores the need to understand how to wisely quantify material losses due to long-term climate change of which this research will focus on.

So far, not many studies have attempted the use of the available dose-response functions that enable a direct link between environmental and atmospheric pollution parameters to long-term material loss, which offers a great opportunity to take into account the effects of potential changing environmental conditions brought about by climate change. Through wise and efficient use of these state-of-the-art deterioration models, which capture material loss effects, there is the opportunity of extending such analyses and linking them to long-term performance modelling of structural elements. This study therefore, seeks to look at the effects of long-term changes in environmental conditions, brought about through climate change, on the deterioration of carbon steel used for metallic bridge structural components such as plate girders and quantify the potential changes in the performance characteristics of such structural elements, focusing on steel plate elements. The research focuses on atmospheric corrosion as a result of environmental exposure and how this will affect long-term structural behaviour and performance.
1.3 Aim and Objectives
The aim of this research is to investigate the potential effects of climate change on material degradation and buckling resistance of steel plate elements, with a view to quantify the long-term performance trends of such structural elements under the changing environment.

To accomplish this aim, the following objectives are set:

- To review the state of the art deterioration models capable of capturing the interaction between environmental and atmospheric pollution parameters and material deterioration for carbon steel used in steel bridges to enable their long-term corrosion predictions.
- Propose a pathway for buckling performance assessment of steel plate elements used in bridges under varying levels and patterns of atmospheric corrosion.
- Propose finite element models for buckling strength assessment of deteriorated steel plate elements used in metallic bridge construction.
- Quantify the effect of varying levels and patterns of atmospheric corrosion on plate buckling resistance.
- Quantify the potential effects of climate change, under different future climate scenarios, on buckling strength of steel plates over time.

1.4 Research Scope and Thesis Overview
This thesis is organized into seven chapters, as follows:

Chapter one sets the introductory background to the research problem and presents explicitly the aim and objectives of this research as well as its scope.

Chapter two presents a review of background theory and relevant literature bridging climate change, long-term material degradation and structural performance, focusing on steel material. The history, classification, behavior and failure mechanisms of steel bridge elements are reviewed, with particular emphasis given to the buckling behaviour of plate elements of steel girders.

Chapter three presents benchmark analyses and verification studies carried out to propose appropriate finite elements for capturing performance assessment of plate elements. Relevant theories used to support the research are also discussed.
Chapters four and five present the finite element analyses of steel plate elements under different corrosion scenarios under compressive and shear loads, respectively. The results offer a full mapping of the buckling capacities of plate elements under different corrosion intensities and patterns.

Chapter six quantifies the potential effects of climate change, through different scenarios, on the long-term buckling resistance of plate elements. The bridging of the two is carried out through the use of the dose-response functions.

Chapter seven includes an overall summary, the main conclusions and recommendations arising from this research work. Suggestions for further studies are also proposed.
CHAPTER TWO

Literature Review and Background Theory

2.1 Introduction
This chapter reviews relevant literature and background material related to this research. The chapter will first look at steel bridges, their history, classification, structural components and behaviour. Particular focus will be given to plate girders and their plate elements associated with their buckling behaviour. Secondly, the thematic subject of climate change will be reviewed and its link with the long-term deterioration of engineering assets such as steel bridges will be highlighted. The relevant climatic and atmospheric parameters that affect steel deterioration will be discussed along with the processes involved. The concepts and definitions of environmental corrosivity and how materials are classified together with the dose response functions, which offer the explicit link between material deterioration and climatic and atmospheric pollution variables, will be discussed. Furthermore, the implication of deterioration on whole-life-cycle performance of engineering infrastructure will also be discussed.

2.2 Steel Bridges
Bridges play an important role in transport infrastructure throughout the world (Caglayan et al., 2009). Since the start of civilisation bridges for roads, railways, walkways and canals have been built, therefore it is impossible to conceive today’s world without bridges (Biezma and Schanack, 2007). One of the main construction materials for bridge structures is steel. Use of structural steel in bridges exploits steel’s advantageous properties of economically carrying
huge loads over long spans with minimum dead load (Hayward, 2003). Steel is suitable for all span ranges as presented in Table (2.1).

**Table 2.1:** Span ranges of bridges

<table>
<thead>
<tr>
<th>Span</th>
<th>Ranges (m)</th>
<th>Bridge type suitability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Short</td>
<td>Up to 30</td>
<td>Railway bridges</td>
</tr>
<tr>
<td>Medium</td>
<td>30 to 80</td>
<td>Composite highway bridges</td>
</tr>
<tr>
<td>Long</td>
<td>80 above</td>
<td>Cable stayed bridges, suspension bridges, footbridges</td>
</tr>
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Most metallic bridges are made of carbon steel and low alloy steel (i.e. steel with carbon content in the range of 0.12 to 2.0% as the main alloying constituent) due to the competitive initial cost advantage of this material over others like aluminium and stainless steel products (Gardner *et al.*, 2007). Though the later material tends to be gaining recognition recently owing to its whole-life-cycle cost advantage over time but still a large stock of existing metallic bridges are made of carbon steel.

### 2.2.1 Historical review of iron and steel bridges

Steel has a long history and future in bridge construction. Some of the early iron and steel bridges are still in use today for example, the Anvil footbridge, Basingstoke, UK (Figure 2.1) and the 30m span Coalbrookdale iron arch bridge, Telford (Figure 2.2) which is one of the world’s first cast iron bridge built in 1779 and still carries occasional light transport and pedestrians (Dietrich, 2001). Up until 1840 the main construction material was either cast iron or wrought iron or a combination of both, but wrought iron replaced cast iron in the early 1800s; many of the early railway bridges were built utilising riveted wrought iron construction. However, in the late 1800s steel began to replace wrought iron and by the early 1900s wrought iron became obsolete as worldwide steel manufacturers moved towards
producing carbon steel which at that time was a much more reliable material. As a result, today steel is one of the main bridge construction material (Biezma and Schanack, 2007).

The history of steel bridge construction cannot be complete without mentioning the Forth Bridge in Edinburgh, Scotland. This bridge was designed by Benjamin Baker and built by William Arrol. The bridge sustains two main spans of 518m. At its time of construction it was the world’s longest spanning bridge with a steel superstructure. It is still in use today on the main Edinburgh to Aberdeen line (Figure 2.3).
The introduction of automatic welding in the mid-1900s brought major changes to the steel fabrication industry (Clarke & Coverman, 1987). In some countries however it took until the 1960s before riveted construction gave way to bolted and welded construction.

Figure 2.3 Forth Bridge, Edinburgh, Scotland, UK (www.allposters.co.uk/Neale-Clarke)

From the 1930s, notable large steel bridges that were built were:

- George Washington suspension bridge, USA (1931) Figure 2.4
- Sydney Harbour Bridge, Australia (1932) Figure 2.5
- Golden Gate suspension bridge, San Francisco, USA (1937) Figure 2.6
- Severn Bridge, Chepstow, UK (1966) Figure 2.7
The period between 1950s to the 1980s constituted the UKs main motorway construction of which most favoured the use of concrete. However, from the 1980s UK fabricators invested hugely in machinery in order to reduce costs. The arrival of heavy lifting equipment allowed steelwork to be erected far more quickly. This, together with the need to construct in restricted conditions made steel far more competitive.

The use of steel offers a wide range of advantages, not only from the material side of view, but also from its broad architectural possibilities, examples of which are given below:
- High strength to weight ratio
- High quality material
- Versatility
- Speed of construction
- Durability
- Modification and repair
- Aesthetics and
- Recycling

The high strength to weight ratio of steel minimises substructure cost which is particularly advantageous in poor ground conditions. Minimum self-weight is an important factor when looking at transporting and handling of components. Steel as a high quality material is readily available in various certified grades, shapes and sizes. The testing regime is done in the steel mills which gives confidence to both clients and engineers who use and specify steel in projects. Prefabrication in controlled conditions also enhances steel quality with a resultant effect on work quality at minimum cost. Prefabrication of components also means expedite construction time even in difficult conditions. Because of this high speed of construction, reduced period of rail possession and road closures is achieved, thus minimising disruption to public usage of such networks. In some circumstances, the light-weight nature of steel can permit complete bridge installation overnight. Steel fits a range of construction methods and sequences. Therefore, installation may be carried out by cranes, launching, slide-in techniques or transporters. With steel the contractor has flexibility in terms of erection sequence and programme. Steel bridges are adaptable, therefore, can readily be modified for a change in use. They can be widened to accommodate extra lanes of traffic, and strengthened to carry heavier traffic loads. Steel bridges can be repaired after accidental damage and at the end of their useful life they can be recycled, making steel a ‘sustainable’ material. This benefit of ease to repair is highly valuable to infrastructural owners and managers, considering at the same time planning of the costs. This research aims at proposing strength reduction indices for bridge elements with respect to long-term deterioration; such information can guide infrastructural managers towards planning maintenance needs for steel bridges arising from deterioration.
Figure 2.6 Golden Gate suspension Bridge, San Francisco, USA (www.lifefoc.com)

It should however be noted that, despite all that is said about the beneficial aspects of using steel for bridges, successful history of steel bridges has not and is not without catastrophes (Preuß 1994). According to Biezma and Schanack (2007), each catastrophe has various causes. These causes may be multiple in most situations of bridge collapse, but each collapse situation has a main cause of which they can be classified according to the following main criteria:

- Force majeure (avalanche, flood, earthquake, terrorist attack etc.);
- Structural and design deficiencies;
- Accidental overload and impact;
- Construction and supervision mistakes;
- Scour and
- Lack of maintenance and inspection.

This study focus on the aspect of structural deficiencies related to material deterioration and will link that to potential changes in environmental conditions to understand how this may impact on long-term resistance loss and hence maintenance planning of such infrastructural assets.
It should be noted that, every bridge exists in an environment, and in this environment of existence these assets suffer from atmospheric corrosion due to exposure to the environment (Kayser and Nowak, 1989). The long-term deterioration of metallic bridges is triggered by this exposure condition which is defined by environmental and atmospheric parameters like relative humidity, temperature, pollutants, time of wetness and atmospheric pollutant concentrations such as SO$_2$. These parameters affect the deterioration process including the rate of corrosion (Kallias and Imam, 2013). Giving the global challenge of climate change on the environment, it is thought that the environment of existence of these bridges is also changing accordingly which may in turn influence the deterioration rates of structural materials.

Construction materials especially iron and steel are susceptible to weather or other corrosive influences. To prevent deterioration, materials are protected by coating to completely avoid or slow down corrosion. Also, at the design stage, a corrosion thickness allowance is added to compensate for the potential thickness reduction due to corrosion over the service life of the structure (Saad-Eldeen et al., 2013).
2.2.2 Classification of Bridges
In terms of classification, steel bridges can be classified according to:

1. The type of traffic the bridge is to carry
2. The type of structural system used and
3. The position of the carriage way relative to the main structural system.

However, bridges are further divided into:

- Highway or road bridges
- Railway or rail bridges and
- Road-cum-rail bridges.

This research will pay particular attention to the classification based on the main structural system. Therefore, it should be noted that, many different types of structural systems are used in steel bridges depending upon the span, carriageway width and type of traffic. Hence, classification according to makeup of the main load carrying system is explore further.

In terms of load carrying system the following bridge types can be outlined.

1. Girder bridges
2. Rigid frame bridges
3. Arch bridges
4. Cable stayed bridges and
5. Suspension bridge

2.2.3 Girder Bridges
Girder bridges predominate when it comes to short and medium spans bridges. They generally provide the most economical solution (Hayward, 2003). For simple spans up to 25m and continuous spans up to 33m, available rolled universal sections are economical up to 1000mm deep. Very little fabrication is required, usually only for the fitting of stiffeners for bearings and attachment of bracing. However, for longer span bridges, deeper girders are required, therefore, plates are fabricated together to form built-up sections called plate girders. Plate girders are used to carry high vertical loads over long spans for which the resulting bending moment are greater than the moment of resistance provided by available rolled sections. The plate girder as mentioned is a built-up section made of two plate flanges welded to a web plate.
to form an I-section (Roberts and Narayanan, 2003). This fabrication flexibility gives opportunity to vary the web and flange sizes and the steel grade for an efficient design. Therefore, plate girders can be built to any size to satisfy design requirements, but the member proportions are limited by web buckling (Clarke & Coverman, 1987). For economy, variable depth girders are often used, which also enhances aesthetics. In terms of function, primarily the top and bottom flange plates resist the axial compressive and tensile forces caused by the applied bending moments while the web resist mainly the shear. It should be noted that this division of structural action is used as the basis for design in some codes of practice (Roberts and Narayanan, 2003). Plate girders are used in road and railway bridges, and occasionally in buildings where large loads and/or long spans are required. As mentioned, the resulting bending moment due to the vertical loads can be greater than the resisting moments, therefore, for a given bending moment the required flange areas can be reduced by increasing the distance between the flanges (web depth) to ensure economical design. Furthermore, to ensure that the self-weight of the girder is kept to a minimum, the web plate thickness is reduced as the depth is increased, but this may lead to web buckling. This is why web buckling is given more significance in plate girder design than for rolled sections and this is consistent with Clarke & Coverman (1987). The theoretical background behind buckling behaviour of plate elements, which are widely used in steel bridges, is presented in subsequent sections.

2.3 Steel Material Deterioration

Why do materials deteriorate? Seems to be a good question to ask before handling the issue of degrading resistance. It is clear that the driving force for material deterioration is related to its thermodynamic stability with respect to its environment of existence. Thermodynamics is the study of heat in changing physical and chemical processes (Mulheron, 2012). Thermodynamics helps in indicating which material will be inherently stable with respect to a particular environment. Therefore, from the durability perspective, materials that are thermodynamically stable in a given environment cannot deteriorate and will remain forever untouched by chemical attack provided the environment does not change. But environments do change, more so with the challenge of global warming due to pollution. Therefore, a structure that was previously stable may become vulnerable to deterioration. Small changes in the climate surrounding a structure can result in ‘localised’ deterioration that can have
significant serviceability impact. Steel suffers deterioration when its surface is allowed to be wetted (BS EN ISO 9223, 2012) as shown in Figure (2.8) below.

![Figure 2.8 Wet Corrosion on Plate Girder, Johnston RI, USA](image)

**2.3.1 Atmospheric corrosion**

Corrosion is seen as the destruction or deterioration of a material because of reaction with its environment (Fontana & Greene, 1978). Almost all of the general types of corrosion attack take place in the atmosphere. Since in the atmosphere the corroding metal is not covered in large amount of electrolyte, most atmospheric corrosion occurs in highly localized corrosion cells, sometimes producing patterns that are difficult to explain. Corrosiondoctors (2010) argues that because of this potential difficulty, estimation of the electrode potentials on the basis of ion concentration, the determination of polarization characteristics, and other electrochemical operations are not as easy as it will be in aqueous corrosion. However, all the electrochemical factors that are significant in a corrosion process do operate in the atmosphere. Atmospheric corrosion is surely the most visible of all corrosion processes, e.g. rusty bridges, flag poles, buildings and outdoor monuments. It is an electrochemical process which proceeds only in the presence of a liquid phase on the metal and it is further controlled by the availability of oxygen which is in abundance in the atmosphere (Roberge, 2008). Controlling atmospheric corrosion is a very significant task to be done given the colossal economic loss that accompanies it. Atmospheric corrosion accounts for the disappearance of a significant portion of metals produced e.g. agricultural machinery, steel structures, fences,
exposed metals on buildings, automobile mufflers and a myriad of other metal items which are discarded when they become unusable as a result of corrosion. All of these constitute direct loss from corrosion (Corrosiondoctors, 2010). Two mechanisms contribute towards corrosion; direct oxidation and aqueous corrosion.

2.3.2 Factors causing degradation
A large number of factors may be responsible for degradation ranging from weathering and biological factors to stress and compatibility factors. The most significant factor in atmospheric corrosion, overriding the presence of surface contaminants, is moisture in the form of dew, rainfall, melting snow, condensation or high humidity (Corrosiondoctors, 2010). In the absence of moisture, most pollutants would have little or no corrosive effects. As good as rain is to corrosion process, it has a beneficial effect in washing away atmospheric contaminants that have settled on exposed metal surfaces. In the event the rain collects in pockets, crevices or ponds, it accelerates the corrosion process by providing continued wetness (Roberge, 2008). Corrosiondoctors (2010) observes that in the case of dew and condensation they are undesirable in that a film of it saturates with sea salts or acid sulfates and acid chlorides of an industrial area to provide aggressive electrolyte which promotes corrosion except if accompanied by constant rain which will help in washing and diluting or eliminating the contamination. In humid tropics with experiences of night condensation appearing on surfaces, the stagnant moisture film either becomes alkaline when it reacts with metal surfaces or becomes dilute acid when it picks up CO₂ (Corrosiondoctors, 2010).

2.3.3 Types of corrosion

2.3.3.1 Uniform corrosion
This is also known as general corrosion and it is the most common type found in metallic surfaces, see Figure 2.9. It involves a uniform etching of the metal proceeding at almost identical rate upon the whole surface. The general loss of material surface leads to gradual thinning of members and a potential risk of structural failure. From the view point of corrosion inspection, uniform corrosion is relatively detectable and its effect predictable hence it is seen as less troublesome compared to other forms of corrosion, except when corroding material is hidden (Roberge, 2008). In dealing with the effect of uniform corrosion in design, a corrosion allowance is suggested to be provided based on possible thickness loss.
2.3.3.2 Pitting corrosion
This is an extremely localised corrosion process in which deep pits are formed upon the metal surface while the surrounding surface remains without observable attack (Hanson and Parr, 1965; Rahgozar and Smith, 1997), see Figure 2.10. The holes may be small or large in diameter but in most cases they are relatively small. Generally, the pit surface diameter is about the same with the depth or less. Fontana & Greene (1978) describe pitting as one of the most potentially destructive forms of corrosion because it is restricted to a very small area and pit extension into the metal may be difficult to detect. In addition, the pits are also covered with corrosion products and it appears difficult to measure quantitatively and to also compare the extent of pitting because of the varying depths and numbers of pits that may occur under identical conditions. Pitting is also said to be difficult to predict by laboratory testing. Sometimes pitting requires a long time – several months or a year to show up in actual service. Because it is a localised and intense type of corrosion, it is particularly characterised by destructive behaviour with failure occurring with extreme suddenness. The pits usually grow in the direction of gravity i.e. developing and growing downwards from horizontal surfaces. Only rarely do pits grow upwards from the bottom of the horizontal surface. Novák (2007) stated that this type of corrosion is typical for stainless steel and aluminium. It is a
cause of concern especially in high stress regions of the structure since they cause local stress concentrations (Rahgozar and Smith, 1997).

2.3.3.3 Crevice corrosion
Known also as contact corrosion, crevice corrosion describes corrosion in hidden parts or narrow crevices or slots between metals. It occurs at regions of contact of metals with metals or metals with non-metals. The dimension at the slot orifice is usually very small, less than 10µm, but it allows the electrolyte to go inside the crevice which afterwards relates with the electrolyte outside. Because of the small dimension of the slot, convection is impeded and diffusion limited (Fontana & Greene, 1978). It is an intense localised form of attack within crevices and other shielded areas and it is common on steel bridge structures at bolted connections, joints, construction crevices, thread connection, pores of welds, locations where a weld is not continuous, at the edge or riveted joints, under sealing, under deposits, under corrosion products or under disbonded coatings (Novák, 2007).

2.3.3.4 Galvanic corrosion and concentration cells
This is also known as two–metal corrosion, it occurs when a potential difference exists between two dissimilar metals when immersed in a corrosive solution. Corrosion of the less corrosion–resistant metal increases while attack of the more resistant material decreases as
compared with the behaviour of these metals when they are not in contact. The less resistant metal becomes anodic while the more resistant metal cathodic (Fontana & Greene, 1978). This phenomenon is experienced when dissimilar metals are electrochemically coupled at bolted or welded connections (Fontana & Greene, 1978) and where two or more areas of metallic surfaces are in connection with different concentrations of the same solution leading to development of concentration cells like in steel piers and tower legs submerged in seawater (Novák, 2007).

2.3.3.5 Environmentally induced cracking
This is a degradation mechanism that causes sudden failure of metallic materials often with high strength and corrosion resistance. It results from straining of the metal below its yield strength or fatigue strength even in environments with low corrosive aggressiveness. Cracking of the metal occurs from the tensile component of the stress which is noticeable. As for the compressive component it does not cause the attack. Novák (2007) outlined three types of cracking that can develop in an aqueous solution; environment including stress corrosion cracking, corrosion fatigue and hydrogen-assisted cracking.

2.3.3.6 Corrosion fatigue
This type occurs when cyclic stresses and corrosion are present in a member. It is a form of attack which requires co-operation of a corrosive environment and cyclic mechanical stress with a tensile factor. Novák (2007) states that, in cyclic stress, the metal integrity is violated by crack development even if it is stressed below yield strength without any corrosive environment. But in corrosion fatigue, the cracks are transcrystalline and propagate discontinuously resulting in striations development.

2.3.3.7 Stress corrosion cracking
Stress corrosion cracking results when static tensile stress affects a metallic material exposed in a specific environment (Fontana & Greene, 1978). In other words, it is the result of the simultaneous effect of tensile stress and specific corrosive environment on a member. It is usually negligible in mild and carbon steel bridges in ordinary environments.
2.3.3.8 Other forms of corrosion
Other forms of corrosion that exist include intergranular corrosion, filiform corrosion, fretting corrosion, erosion corrosion, dealloying, hydrogen damage and microbial corrosion. These will not be discussed in this report as they are out of the scope of the current investigation.

2.3.4 Corrosion Environments

2.3.4.1 Atmosphere
The atmosphere happens to be the environment of existence of constructional systems. Many metallic surfaces and structural components are exposed to corrosion effect of outdoor atmosphere. Similar to the corrosion of metals in electrolyte, atmospheric corrosion is an electrochemical mechanism at normal temperatures. The interaction of dry air with construction metals at normal temperature is negligible; atmospheric corrosion occurs only due to the presence of atmospheric humidity (Novák, 2007; Knotkova and Kreislova, 2007). This humidity forms a sufficiently thick electrolyte film necessary for the corrosion process at critical humidity. This condition is met when relative humidity exceeds a critical value of 60-80% (10-14g H₂O/m³ of air at 20°C). At sub-critical humidity, the corrosion rate is not zero, but for majority of technical application of metals it is insignificant. The corrosion effect of an atmosphere in a specific location is given by the time for which the relative humidity of the atmosphere is being influenced by stimulators like SO₂ and chlorides.

2.3.4.2 Aqueous solution
Principally, here we refer to water, not only chemically pure water but also various weak-concentrations of aqueous solution containing substances that get into water during its natural and industrial hydrological cycle. Metal surfaces exposed to absolute pure water do not corrode significantly. What makes water aggressive and responsible for the process of corrosion reaction are the additional agents at certain amounts which are both present in natural and industrial waters such as oxygen, aggressive gases, dissolved salts, organic substances, microorganisms, pH, temperature flow rate, and content of solid particles (Moncmanová, 2007).

2.3.4.3 Soil
Soil is one of the environments of existence of metallic structures such as tanks and pipelines which are buried within the soil. Soil aggressiveness to these structures is dependent on the
type and cohesion of the soil, its homogeneity, humidity, chemical composition of soil electrolyte, pH and redox potential, buffer capacity and oscillation of ground water level. Corrosion attack of metallic materials in a soil would be absolutely indistinctive without the presence of humidity (Moncmanová, 2007).

2.3.4.4 Concrete
The required mechanical properties of concrete are usually achieved with the introduction of steel reinforcing bars usually made of carbon steel. In fresh concrete, carbon steel is usually passive because of the developed alkalinity from cement content. This alkalinity ensures spontaneous passivation of the steel bars, therefore slowing the rate of corrosion. But with the ingress of oxygen, carbon dioxide and subsequently chloride from the atmosphere into wet concrete, the oxidising power of the pore solution is increased making the existing environment of the steel reinforcement bars aggressive due to reduced pH. This leads to carbonation which if it gets through to the steel through the concrete cover (usually between 20-30mm) it activates the steel leading to substantial increase in the corrosion rate (Moncmanová, 2007).

2.3.5 Corrosivity of atmospheres; classification, determination and estimation
Corrosivity of atmosphere defines how corrosive or aggressive an atmosphere is i.e. it spells out the ability of an atmosphere to cause corrosion in a given corrosion system (BS EN ISO 9223, 2012). This also relates to the properties of the so-called corrosion system in that the character of the attack as well as the rate of corrosion depends on the system. The following factors characterise the system (BS EN ISO 9223, 2012; Knotkova and Kreislova, 2007).

- The material i.e. metal
- The atmospheric environment i.e. temperature-wetness complex, TOW and level of pollution
- Technical parameters i.e. shape, weight, construction design, joints, treatment type
- Condition of operation.

Though these four factors determine the corrosion attack and its rate, Albrecht and Hall (2003) show that the greatest influence on thickness loss of metals is the atmospheric environment. This environment is defined by both climatic factors and pollutants (Pourbaix and Miranda, 1983).
When materials are exposed to the atmospheric environment they become susceptible to deterioration. It is information about the corrosivity characteristics that provides the basic requirement for material selection, provision of protective system, estimation of service life and evaluation of damage caused by corrosion.

Environmental pollution has a significant contribution to material degradation. Once a material enters a polluted environment the degradation process is accelerated, though the process differs for different materials depending on their composition and other characteristics. The majority of materials exposed to atmospheric environment are sensitive to pollutants and the acidity of precipitation. The corrosion rate is strongly influenced by the air pollution while the process occurs due to humidity (Knotkova and Kreislova, 2007).

2.3.5.1 Classification of corrosivity
BS EN ISO 9223 (2012) classifies atmospheric corrosivity into two categories:

- Classification by determination of corrosivity based on clause 7. This emphasises determination of corrosivity based on corrosion rate measurement on standard coupons.
- Classification based on estimation in accordance with clause 8 which emphasises estimating corrosivity based on environmental information.

2.3.5.2 Classification based on characterisation of atmosphere
Corrosivity here is estimated based on three environmental parameters: time of wetness, sulphur compounds and airborne salinity contamination i.e. corrosivity is based on corrosion loss calculation from environmental data or information on environmental conditions and exposure situation (BS EN ISO 9223, 2012; Knotkova and Kreislova, 2007).

2.3.5.3 Classification based on measurement of corrosion rate
This is a corrosivity classification approach based on the determination of corrosion rate of standard metals specimens. Here, the influence of pollution on corrosion rate of basic construction metals is systematically observed first for a long-term. Afterwards deterioration functions that express the dependence of the corrosion rate on environmental factors like temperature, precipitation, relative humidity, sulphur dioxide, chlorides are developed. These functions, which are generally called dose-response functions and express the dependence of the degree of deterioration on pollution and other environmental parameters, can be transformed into a service life model. The derivation of such function seems to be the major
contribution to the improvement of the classification system. Using the classification system can help in fully evaluating environmental stress, selection of anticorrosion measures and the estimation of service life of components and structures (BS EN ISO 9223, 2012; Knotkova and Kreislova, 2007). One the biggest potentials that dose-response function also offer is the capability of taking into account the effect of changing environmental and atmospheric pollution parameters on long-term material deterioration. Dose-response functions will be discussed in detail in the following sections.

2.3.6 Using new knowledge to improve classification system
Data from the international exposure programs ISOCORRAG and MICAT from tropical and cold regions contributed significantly in the adjustment of the existing corrosivity classification. It helped in deepening the knowledge of the effect of individual environmental factors and groups of these factors on the kinetics of the corrosion process (Knotkova and Kreislova, 2007). The works of Tidblad et al. (1999), Tidblad et al. (2002) and Morcillo, Almeida and Rosales (1998) elaborated further on the results from these programs. The basic contribution was the development of new and better elaborated dose-response functions that are generally valid for a wide regional scale. These functions are all based on data after one year of exposure and can therefore, only be used for classification purposes and not for assessing service life of materials in different environments (Knotkova and Kreislova, 2007). The dose-response (D/R) functions for estimating the first year corrosion loss of four structural metals are given below (Knotkova and Kreislova, 2007):

Carbon steel:

\[ r_{corr} = 1.77.D_d^{0.52}.\exp(0.020.RH+f_{st})+0.102.S_d^{0.62}.\exp(0.033.RH+0.040.T) \]  \hspace{1cm} (2.1)

\[ f_{st} = 0.150(T-10) \text{ when } T \leq 10^\circ C, \text{ otherwise } -0.054(T-10) \]

Zinc:

\[ r_{corr} = 0.0129.D_d^{0.44}.\exp(0.046.RH+f_{zn}) + 0.0175.S_d^{0.57}.\exp(0.008.RH+0.085.T) \]  \hspace{1cm} (2.2)

\[ f_{zn} = 0.038(T-10) \text{ when } T \leq 10^\circ C, \text{ otherwise } -0.071(T-10) \]

Copper:

\[ r_{corr} = 0.0053.D_d^{0.26}.\exp(0.059.RH+f_{cu})+0.01025.S_d^{0.27}.\exp(0.036.RH+0.049.T) \]  \hspace{1cm} (2.3)
Climate Change Effects on Buckling Strength of Steel Plate Elements

\[ f_{cu} = 0.126(T-10) \] when \( T \leq 10^oC \), otherwise \(-0.080(T-10)\)

Aluminum:

\[ r_{corr} = 0.0042.P_d^{0.73}\exp(0.025.RH+f_{al})+0.0018.S_d^{0.60}\exp(0.020.RH+0.094.T) \]  

(2.4)

\[ f_{al} = 0.009(T-10) \] when \( T \leq 10^oC \), otherwise \(-0.043(T-10)\)

Where \( r_{corr} \) = 1st year corrosion rate of metal in \( \mu m/\text{year} \), \( T \) = temperature in \( ^oC \), \( RH \) = relative humidity in \( \% \), \( P_d \) = \( \text{SO}_2 \) deposition in \( \text{mgm}^{-2}\text{day}^{-1} \), \( S_d \) = \( \text{Cl}^- \) deposition in \( \text{mgm}^{-2}\text{day}^{-1} \)

2.3.7 Dose-Response Functions (DRFs)

Dose-response functions are models that are developed to aid in the determination of corrosion loss of metals and alloys. They are proposed as a variant of improving the assessment of the corrosivity of outdoor atmosphere from the environmental parameters (Jernberg et al., 2004; Mikhailov, Tidblad and Kucera, 2004). Over the years, several models have been developed by researchers. The earlier models predict corrosion loss as a function of time (Klinesmith, McCuen and Albrecht, 2007; Melchers, 2003) since for steel, corrosion rate rises to a maximum and then gradually reduces with time due to build-up of corrosion products (Barton et al., 1980). Other researchers argue that corrosion rate follows a logarithmic curve (Pourbaix and Pourbaix, 1989). Whatever the argument, the variation of atmospheric corrosion was initially expressed as a power function in the form

\[ M=Kt^n \]  

(2.5)

Where \( M \) is corrosion mass/thickness loss per unit of exposed area, \( t \) is exposure time, \( K \) is a proportionality constant and \( n \) is mass loss exponent.

In this model, \( K \) and \( n \) are empirical coefficients determined by a log-linear regression analysis of the measured data. Because equation (2.17) is a time dependent model, it means \( K \) and \( n \) represents the effect of all other factors that cause the corrosion process such as environmental conditions. This is the obvious setback of such models when used for predictions, the results will be inaccurate giving the obvious reason and the fact that \( K \) and \( n \) will reflect the condition of the calibration environment (Klinesmith, McCuen and Albrecht, 2007).

According to Benarie and Lipfert (1986), equation (2.17) is a mass-balance equation that the corrosion process is rate determined depending on the diffusion ability of the properties of the
layer separating the reactants. Dean and Reiser (2002) show that environmental parameters like TOW positively affect the values of the mass exponent (n) for steel, while SO$_2$ and Cl do not show significant effect on slope (K) but on the intercept (n). These two researches further proof the setback of equation (2.17).

This reality opened the way for the development of models that will somewhat represent other environmental effects (Mikhailov et al., 2004; Klinesmith et al., 2007; Mikhailov et al., 2007; Granata et al., 1996).

According to Panchenko and Strekalov (2001), any good engineering design should look at the environmental exposure condition of the intended structure, because depending on the climate and corrosivity, such a structure undergoes aging which affects its durability which in turns affects structural performance. Stressing further, they said that atmospheric resistance of metals defines the ability of the metal to resist extreme atmospheric condition; this is consistent with Landolfo et al. (2011). ISO 13823:2008 (2012) defines durability as the capability of a structure and its components to satisfy planned maintenance and the design performance requirement over a specified time period under the influence of environmental actions. This definition is consistent with (Grondin et al., 2004). Granata et al. (1996) opined that higher rates of steel degradation are caused by exposure to aggressive environmental situation which are a mixture of both natural and manmade conditions, with a resultant effect on durability and longevity of structures.

2.3.7.1 Dose- response function developmental stages

The developmental stages of the dose-response functions are the following three:

1. The formulation stage
2. The calibration stage
3. The validation stage

These three stages are aimed at achieving a model that is a representation of the structure in a theoretical rational manner being calibrated using environmental coefficients that are relevant to it with accurate validity for the environment having been tested with independent data (Klinesmith et al., 2007). Mikhailov et al. (2004) express the three component parts that make up the general corrosion model with environmental factors effects as

\[ C = f_{\text{dry}} (\text{SO}_2) + f_{\text{dry}} (\text{Cl}^-) + f_{\text{wet}} (\text{H}^+) \] (2.6)
Where \( C \) = total corrosion effect

\[ f_{\text{dry}} (\text{SO}_2) = \text{effect of dry deposition of sulfur dioxide} \]

\[ f_{\text{dry}} (\text{Cl}^-) = \text{effect of dry deposition of chlorides} \]

\[ f_{\text{wet}} (\text{H}^+) = \text{effects of wet deposition of hydrogen ions (acid rain)} \]

For atmospheres containing \( \text{SO}_2 \) such as industrial and urban atmospheres where they make the greatest contribution in atmospheric corrosion:

\[ f_{\text{dry}} (\text{SO}_2) = A (\text{SO}_2)^B (\text{TOW})^C \]  \hspace{1cm} (2.7)

Where \( \text{SO}_2 = \) sulfur dioxide concentration

\( \text{TOW} = \text{Time of wetness} \)

\( A, B, C = \text{constant coefficients} \)

This equation correlates corrosion under an adsorption water film with \( \text{SO}_2 \) absorbed by the film during the time equal to \( \text{TOW} \).

Modifying equation (2.10) and utilizing the \( \text{TOW} \) and \( T \) parameters gives

\[ f_{\text{dry}} (\text{SO}_2) = A (\text{SO}_2)^B (\text{TOW})^C \exp(g(T)) \]  \hspace{1cm} (2.8)

In atmospheres containing \( \text{Cl}^- \) ions e.g. coastal and marine environments, the chlorides stimulate corrosion given that they are hygroscopic in nature. This hygroscopic nature creates the electrolyte layer on the metal thus decelerating evaporation and increasing the time of wetness even at higher temperatures. Increasing temperature in this situation increases corrosion rate.

It is expected that in coastal atmospheres corrosion models are based on using time of wetness and chloride concentration. Therefore:

\[ f_{\text{dry}} (\text{Cl}^-) = A (\text{Cl}^-)^B (\text{TOW})^C \]  \hspace{1cm} (2.9)

Attempts were made to capture the effects of temperature in this equation but because both experimental as well as statistical data were limited, the attempts were not successful. However, using the ISO CORRAG program data, for the first time temperature was
successfully represented as a growing exponential in the corrosion rate model for carbon steel in coastal atmospheres as follows:

\[ f_{\text{dry}} (\text{Cl}^-) = D \text{Cl}^E \text{TOW}^F \exp (KT) \]  
(2.10)

Where Cl = chloride dry deposition

TOW = Time of wetness

T = Temperature and

D, E, F and K = constants

This equation was shown to be valid for other metals as well.

The total corrosion effect here is represented by the product rather than the sum of the function of all the three parameters.

When using T and Rh in place of TOW, the model to represent the equation is:

\[ C = A_1 (\text{SO}_2)^{B_1} \exp (C_1 (\text{Rh}) + g_1 (T)) + D_1 \text{Cl}^E \exp (m_1 (\text{Rh}) + K_1 (T)) \]  
(2.11)

Hence, the dose response function developed is as follows:

\[ C_{\text{st}} = 0.085 (\text{SO}_2)^{0.56} (\text{TOW})^{0.53} \exp (f_{\text{st}}) + 0.24 \text{Cl}^{0.47} (\text{TOW})^{0.25} \exp (0.049T) \]  
(2.12)

With \( f_{\text{st}} (T) = 0.098 (T-10) \) for \( T < 10^\circ \text{C} \) and

\( f_{\text{st}} (T) = -0.087 (T-10) \) for \( T > 10^\circ \text{C} \)

2.3.7.2 A critical review of the dose-response functions

Guttman and Sereda (1968) developed models that relate corrosion loss \( K_o \) to time of wetness (TOW) and SO\(_2\) concentration as follows

\[ K_o = 0.16 \text{TOW}^{0.7} (\text{SO}_2 + 1.78) \]  
(2.13)

Assuming that SO\(_2\) concentration becomes zero given that it does not exist in the environment, equation (2.13) becomes

\[ K_o = 0.285 \text{TOW}^{0.7} \]  
(2.14)
By looking at equation (2.14), we can see that it can estimate corrosion loss as a function of TOW only. This equation is limited in predicting corrosion over service life given that $K_o$ is only a function of TOW and time i.e. a relationship between TOW and exposure period over which wetness occurs. While service life concept presupposes that a structure should be evaluated in terms of its structural, environmental and economic performance during its whole-life cycle (Landolfo et al., 2011).

Looking at the work of Hakkarainen and Ylasaari (1982), their dose-response function combines TOW with SO$_2$ concentration as follows

$$K = 1.17 \text{TOW}^{0.66} (\text{SO}_2 + 0.048) \quad (2.15)$$

It is evident that if SO$_2$ is made zero, equation (2.15) becomes

$$K = 0.0562 \text{TOW}^{0.66} \quad (2.16)$$

Comparing equation (2.15) and (2.16) reveals that the proportionality constant of equation (2.14) is greater than that of (2.16) by a factor of five or more, but their time of wetness exponents are almost similar in value. The difference is indicative of the magnitude of variation in corrosion loss for locations with different environments.

Haynie and Upham (1974) developed models that relate corrosion penetration $K$ to SO$_2$ concentration, percentage relative humidity (Rh) and exposure time:

$$K = 325t^{0.5} \exp (0.00275\text{SO}_2 - (163.2/Rh)) \quad (2.17)$$

The corrosion loss is determined as a product of a time-dependent power function and an exponential function. Obviously, this equation uses relative humidity as a factor to predict corrosion loss, while several other models use TOW given that it accounts for the relative humidity.

The models presented by Attaraas (1978), cited in Klinesmith et al. (2007), estimate corrosion loss for one-year ($K_1$) and for four-year ($K_4$) periods. The models utilise only SO$_2$ concentration as seen below:

$$K_1 = 2.28\text{SO}_2 + 176.6 \quad (2.18)$$

$$K_4 = 18.5\text{SO}_2 + 292.5 \quad (2.19)$$
These models are similar to those developed by Knotkova (1984) cited in Klinesmith et al. (2007), presented below, which were also based on SO$_2$ deposition:

\[ K_1 = 4.0SO_2 + 58 \] (2.20)

\[ K_4 = 8.0SO_2 + 168 \] (2.21)

With further extension of the models to reflect the effect of air salinity (Cl), TOW and SO$_2$ pollution:

\[ K = 1.327 + 0.4313SO_2 + 0.0057TOW + 0.138Cl \] (2.22)

Considering equations (2.18), (2.19) and (2.20), (2.21) and assuming that SO$_2$ is set to zero, we obtain (in g/m$^2$)

\[ K_1 = 176.6 \quad \text{and} \quad K_4 = 292.5 \]

\[ K_1 = 58 \quad \text{and} \quad K_4 = 168 \]

From the above, we can deduce the following conclusions:

1. Corrosion loss must be equivalent of these values or higher
2. Item (1) limits the flexibility of the models
3. The effect of TOW is ignored in calculating corrosion loss.

From the corrosion loss values of (2.20) and (2.21), Klinesmith et al. (2007) observed that the values are significantly less than what they should be for the same exposure times for equations (2.19) and (2.20).

The one-year exposure, $K_1$, given by equation (2.18), is greater than 1/3 of $K_1$ given by equation (2.20). The four-year exposure, $K_4$, given by (2.19), is almost half of $K_4$ given by equation (2.31b). Klinesmith et al. (2007) concluded that the difference in corrosion loss in this analysis highlights the associated problems with using empirical models at sites where they were not originally calibrated.

From equation (2.32), the corrosion loss is estimated as a function of SO$_2$, TOW and air salinity (Cl) as mentioned before. It can be seen that the model is linear in nature thus assuming that the effect of each environmental factor is additive. Corrosion loss here will not
be zero given the constant 1.327 in the equation. Thus it is not a true representation of the process of corrosion.

Analysing the model developed by Barton et al. (1980), cited in Klinesmith et al. (2007), which relates corrosion loss to TOW and SO$_2$ concentration, we have:

$$K = 0.0152 \text{TOW}^{0.428} (\text{SO}_2)^{0.570}$$  \hspace{1cm} (2.23)

The function is a power model with two predictors: TOW and SO$_2$ concentration. When SO$_2$ is set to zero corrosion becomes zero, therefore it is not realistic because many factors contribute to corrosion loss and the corrosion process continues even if SO$_2$ concentration is zero. Therefore, it can be argued that this model does not provide an accurate physical model.

The dose response functions presented earlier were all developed based on different exposure programs ranging from the NAPAP program (Baedecker, 1990) and the Scandinavian exposure program (Haagenrud and Henriksen, 1996) to the 1986 ISO CORRAG program (Mikhailov et al., 2004) and the ISO CORRAG-MICAT 1995 program (Morcillo et al., 1998). It can be seen that the models developed by these exposure programs for steel have one common dominant corrosion parameter, the concentration of SO$_2$. Between 1987 and 1994, additional dose-response functions have been developed from international co-operative programs in which 8-year material exposure tests have been carried out (Tidblad et al., 2001). Recently, as a result of policy, SO$_2$ levels have decreased; however, due to increase in car traffic, nitrogen compounds and ozone particulate levels in the atmosphere have been elevated. As a result, a multi-contaminant situation presents threats in Europe where SO$_2$ is no longer the dominant factor affecting material corrosion. The ICP material multi-pollutant exposure program (1996) recognized this. The program was later extended to capture the effects of HNO$_3$ and particulate matter in the MULTI-ASSESS project (Kucera et al., 2005). This knowledge and understanding gave birth to the Kucera et al. (2007) and BS EN ISO 9223 (2012) dose response functions. The MULTI-ASSESS project characterised environments in terms of their climatic parameters, gaseous pollutants and precipitation. In looking at climatic parameters, temperature, relative humidity, time of wetness and sunshine radiation were considered. The precipitation parameters include total precipitation amount, conductivity and concentration of the ions of H$^+$, SO$_4^{2-}$, NO$_3^-$, Cl$^-$, NH$_4^+$, Na$^+$, Ca$_2^+$, Mg$_2^+$ and K$^+$. In addition to these parameters, Kucera et al. (2007) measured the total mass of HNO$_3$ and particulates for the EU 5FP MULTI-ASSESS project.
Kucera et al. (2007) having considered the effects of SO$_2$ concentration, relative humidity, temperature and chlorides in developing models for carbon steel concluded from the statistical evaluation that HNO$_3$ is not of significant influence on carbon steel deterioration. Therefore, the first year corrosion was expressed as a function of the remaining parameters as follows:

$$r_{corr} = 1.77(SO_2)^{0.52}\exp^{0.02\text{Rh}}\exp^{f(T)} + g(Cl^-,\text{Rh},T)$$

(2.24)

Where $r_{corr}$ = corrosion loss in µm

$$f(T) = 0.15\ (T-10)\ \text{ where } T< 10^\circ\text{C};\ \text{otherwise } -0.054\ (T-10)$$

$g\ (Cl^-,\text{Rh},T) =$ function describing the effect of dry deposition of chloride in combination with Rh and temperature.

BS EN ISO 9223 (2012) models were developed considering the effects of temperature (T), relative humidity, SO$_2$ deposition and chloride Cl$^-$ on carbon steel based on a worldwide corrosion field exposure programme covering different climate conditions as well as pollutants. The following model emerged:

$$r_{corr} = 1.77P_d^{0.52}\exp^{(0.02\text{Rh} + fst)} + 0.102S_d^{0.62}\exp^{(0.033\text{Rh} + 0.040T)}$$

(2.25)

where $fst =0.0150\ (T-10)\ \text{ when } T< 10^\circ\text{C};\ \text{otherwise } -0.054\ (T-10)$

$r_{corr}$ = first year corrosion rate in µm/a

T = annual average temperature in °C

Rh = annual average relative humidity in %

$P_d$ = annual average SO$_2$ deposition in mg/m$^2$.d

$S_d$ = annual average Cl$^-$ deposition in mg/m$^2$.d

This is a robust state-of-the-art contribution to the evaluation of corrosion loss shifting from the classical exponential models (equation 2.5) where the constants are determined from different exposure classifications to directly linking environmental and atmospheric pollution parameters to long-term material degradation; this is ideally suited to capture the effects of changing environmental conditions on long-term deterioration, which is the focus of this study.
2.3.8 A contrast between the ISO 9223 and 9224: 2012
As presented above, models in BS EN ISO 9223 (2012) are used to estimate the first year corrosion thickness loss. On the other hand, the BS EN ISO 9224 (2012) models are formulated to determine corrosion loss (D) beyond the first year, as follows:

\[ D = r_{corr} t^b \]  \hspace{1cm} (2.26)

Where \( t \) = exposure time in years

\( r_{corr} \) = first year corrosion rate in \( \mu \text{m/a} \) or \( \text{g/m}^2\text{.a} \)

\( b \) = metal- environment- specific time exponent usually < 1

The above model is used to predict corrosion loss up to 20 years since corrosion product layer increases in thickness with time and its degree of protection during exposure also increases at some point in time above 20 years. Within this first 20-year period, thickness loss is said to be exponential in nature and then it becomes linear thereafter giving rise to the second model, following differentiation of equation (2.26):

\[ \frac{dD}{dt} = b r_{corr} (t)^{b-1} \] \hspace{1cm} (2.27)

Leading to:

\[ D \ (t > 20) = r_{corr} (20^b + b \ (20^{b-1}) \ (t-20)) \] \hspace{1cm} (2.28)

2.4 Structural performance
Structural performance is the ability of a structure or structural part or component to fulfil its function under the intended use condition (Jernberg et al., 2004). This is defined by a performance characteristic i.e. a quantity being a measure of its critical property e.g. strength, thickness loss, etc. Atmospheric corrosion leads to mass loss of the steel section resulting in load bearing reduction due to the section loss as well as notching, stress concentration and possible cracking. The implication of this on structural performance is reduction in resistance area in terms of strength, stiffness and ductility. Once this performance characteristic falls short of the performance criterion, the structural performance is affected. For structural steel, the important properties are tensile and yield strength, fatigue strength, fracture toughness and corrosion resistance (Albrecht and Hall, 2003).

For atmospheric corrosion, the functions that relate the doses of a degradation agent to a degradation indicator are the dose-response functions (DRFs), presented in the previous
sections. These can aid in evaluating the corrosion loss or predicting the corrosion damage of metals and at the same time answering questions concerning durability of metallic structures, economic cost of damages related to the degradation of materials and acquiring knowledge about the effect of environmental variables on corrosion kinetics (Feliu et al., 1993; Feliu and Morcillo, 1993).

Sharifi and Rahgozar (2010c) reported that, the main effect of corrosion on steel structures is material loss which leads to section thinning. In another study by Sharifi and Paik (2009), they reported that corrosion effect is not limited to fracture of structural component alone but also results in the yielding or buckling of members. They showed that a decrease in the net area of a member results in increase in the stresses in the member under the same load. Certain properties of a structural member’s cross section such as section modulus, slenderness ratio, moment of area, radius of gyration can be affected by corrosion deterioration. These properties are critical to the ability of the member in resisting axial forces and bending moments. Member stiffness may also be reduced by section loss which can lead to excessive deflection (Sharifi and Rahgozar, 2010b). Generally, corrosion leads to decrease in the load-carrying ability and reliability of the affected member. Severe levels of corrosion can cause the ultimate capacity of a steel member to fall below service load and failure resulting at a lower load than the design load (Sharifi and Rahgozar, 2010c). The mode of failure of a structural member can also change due to material loss because the member cross-sectional classification may be altered. Sharifi and Rahgozar (2010a) pointed that a plastic section can become semi-compact due to material loss and so local buckling may hinder full development of plastic moment. All of these can also be attributed to corrosion mechanisms due to environmental factors (Kayser and Nowak, 1989a).

2.4.1 Durability
Most recently, actors in the built environment are giving attention to the issues of durability and sustainability in their activities (Landolfo, Cascini and Portioli, 2011). This is not far from the fact of the benefit of optimisation of performance throughout the life time of the structure with respect to environmental, economic and social requirements (Jernberg et al., 2004). Durability concepts are based on the fact that structures deteriorate with time and this deterioration has both serviceability and financial consequences. The concern here is that, why do construction material undergo some form of deterioration with time? On the other hand, sustainability is associated with life-cycle, durability, structural design, ecology and
costing. Therefore, the concepts of durability and sustainability integrate whole-life, time-dependent multi-performance-based design. Loss in performance over time is a clear indication of deterioration taken place. Therefore, service life prediction requires knowledge of the degradation environment and the resistance of materials to factors causing degradation (Jernberg et al., 2004; Sarja et al., 2005). This research is giving special attention to durability design of metallic bridge plate elements with respect to buckling which is capable of impairing the structural performance of these carbon steel components over time.

2.4.2 Deterioration implication on whole-life
Deterioration process is a threat to the life span of a structure. Structures are normally designed to fulfil their purpose within the designed life-time because that is when the benefit of financial investment can be achieved. Sarja et al. (2005) defines life-time engineering as “the theory and practice of predicting, optimising, and integrating long-term investment planning, design, construction and management in use, maintenance, repair and end of life management of assets”. This definition is consistent with Steele et al. (2003b) asserting that a design that considers service and end of life will provide the structure with beneficial operational characteristics. This has an impact on the life-time performance of an engineering system and it is assessed as the structural component’s durability evaluated against the effects of deterioration. The durability performance of metal structures is strongly influenced by damage due to fatigue and atmospheric corrosion and their control is a key aspect for design and maintenance of both new and existing structures (Landolfo et al., 2011). There are a number of factors that lead to corrosion traps (localised corrosion of specific areas) and these factors normally accelerate the corrosion rate. These factors are poor drainage, confined spaces, joints, road salts, microclimates, soluble pollutants, weather condition and general effects. The consequence of their effects is the implication it has on structural integrity or robustness. Giving that these factors influence localised corrosion it is clear that a general corrosion rate cannot be accurately determined. This means that, in most cases, the integrity of any structure is dependent upon the local areas suffering from the highest corrosion rates. Most of the time, the worst affected areas by corrosion are those most difficult to inspect and maintain.
2.4.3 Buckling, Postbuckling and Collapse Behaviour of Plates

Buckling is a failure mode that is characterised by the sudden failure of a structural member subjected to high compressive stresses. It is a nonlinear phenomenon whereby a structure cannot any further take up load with its original geometry and so it changes its shape in order to find an alternative equilibrium position (Sosa et al., 2006). The actual compressive stresses at failure are smaller than the ultimate compressive stresses that the material is capable of bearing (Simulia, 2011). The compressive stresses at failure represent the bifurcation load while the ultimate compressive stresses defines the collapse or limit load (Bushnell, 1981). Interestingly, this phase of the buckling phenomena generally takes place before large deformations occur. At this point the structure appears to be undeformed or just slightly deformed. Buckling occurs when a structural component converts membrane strain energy into bending strain energy with no change in the applied external load (Bushnell, 1981; Simulia, 2011). However, buckling is not necessarily caused by applied compressive forces only, but can also be initiated by other load situations like shear load or bending load. These other load situations create compressive stress conditions within the member that may result in buckling of the member. These compressive stresses result in stress softening, a situation where the member bending stiffness is reduced due to compressive load. For example, if a slender bar of length (L) and cross sectional area (A) is considered, it can be discovered that the axial stiffness of the bar (AE/L) is much greater than the bending stiffness (EI/L^3) of the bar when compared. Hence, any small membrane deformation can absorb a large amount of strain energy. However, comparatively large lateral deflections and cross sectional rotations are needed to absorb this energy in bending (Simulia, 2011). Consider Figure 2.11 which has stiffness $K_a$ equation 2.29

![Slender Bar under Axial Load](image)

**Figure 2.11 Slender Bar under Axial Load (Simulia, 2011)**

\[
K_a = \frac{AE}{L} \quad (2.29)
\]
But knowing that force \((F)\) is proportional to displacement \((U)\) within the elastic limit, therefore

\[ F = KU \]  
(2.30)

Thus we can say,

\[ F_a = K_a U_a \]  
(2.31)

Again, considering Figure 2.12 with stiffness \(K_b\) equation 2.32 and substituting in the definition of force as in equation 2.30 will result in the following.

Figure 2.12 Slender Bar in Bending (Simulia, 2011)

Where stiffness \((K_b) = 48EI/L^3\)  
(2.32)

\[ F_b = K_b U_b \]  
(2.33)

Utilising the definition of work done \((W)\), product of force and distance or displacement, it can be written that

\[ W = F_a U_a = F_b U_b \]  
(2.34)

Simplifying further and substituting for \(F_a\) and \(F_b\) as in (2.31) and (2.33)

\[ K_a U_a U_a = K_b U_b U_b \]  
(2.35)

\[ K_a U_a^2 = K_b U_b^2 \]  
(2.36)

Therefore,
Equation 2.37 illustrate how for bending, a large lateral deflection and cross sectional area will be required to absorb the same amount of strain energy ($W$) that a small membrane deformation will absorb.

### 2.4.3.1 Bifurcation Point

This is a point of critical load where equilibrium becomes unstable. Bifurcation is a phenomena where along the equilibrium path of a structure at a point there is a split into two paths (the primary and the secondary path). Bifurcation, also known as bifurcation buckling, is a discontinuous response at the point of initiation of buckling. The bifurcation point is considered as the load for which a reference configuration of the structure and an infinitesimally close (buckled) configuration are both possible equilibrium configurations (Simulia, 2011). Bifurcation point defines the critical load at which the response of the structure will bifurcate (split into two). At this critical load the structural component bends dramatically and exhibits much lower stiffness. The critical load is a non-conservative estimate of the structure’s load carrying capacity; the actual critical load may be significantly lower than the bifurcation load prediction. The critical load is essentially the load level at which the equilibrium of the structure becomes unstable, the onset of which is predicted by an eigenvalue analysis. The critical bifurcation load is based on the assumption that the structure is perfect. This load is dependent on the stiffness of the structure. Bifurcation is essentially a mathematical concept, in real life it does not exist given that real structural components are not perfect. However, it is a concept that is used to understand the structural behaviour on a load-displacement curve. At the buckling load on the load-displacement path, the deformation begins to grow in a new pattern which is quite different from the prebuckling pattern (Bushnell, 1981). This behaviour can be a linear response up to the bifurcation with postbuckling strength or a nonlinear response up to bifurcation with imperfection sensitive response (Figure 2.13). The primary path defines the path which continues imaginary after the split or division (bifurcation) while the secondary path is the typical postbuckling path after the bifurcation point (i.e. the path showing behaviour after buckling). It should be noted that the engineer’s concern is obviously centred upon the bifurcation point corresponding to the lowest load level because on the basic or primary path there is an infinity of bifurcation points at various load levels. The load corresponding to the lowest load level is referred to as the critical load, as shown in Figure 2.14 (Dubas and Gehri, 1986).
Figure 2.13 Representative load-deflection curves with (a) linear and (b) nonlinear response up to Bifurcation (Simulia, 2011)

Figure 2.14 Possible Equilibrium Paths Showing the Point of Critical Load (Dubas and Gehri, 1986)
2.4.3.2 Limit (collapse) Load
The limit load is considered the maximum load a structural component can take before collapse. It is the relative maximum on the load-displacement curve for which there is no adjacent equilibrium condition. Therefore, the action of the structure at this point becomes dynamic because the slope of the curve becomes negative and, as a result, the structure releases elastic energy which is in turn converted into kinetic energy (Simulia, 2011). The limit load is a function of the material stiffness.

2.4.3.3 Imperfection Sensitivity
All real structures are imperfect, hence they will be sensitive to the relevant imperfections. The level of imperfection principally influences the initial plate stiffness although it is observed that plates with any level of imperfection approach the secondary stable equilibrium path at high strain levels (Dubas and Gehri, 1986). Therefore, an out-of-flatness of the plate with small initial amplitude will exert an influence on the primary equilibrium path. If the initial shape is related to the first mode of instability of the plate, the equilibrium path will follow this shape and merge into the postcritical path of the ideal plate. However, if the initial shape is related to one of the higher failure modes or a combination of these modes which happens not to be in sympathy with the first mode, then the critical point occurs at a higher load than for the corresponding perfect plate (Dubas and Gehri, 1986). As with all steel structures, plate panels contain residual stresses from manufacture and subsequent welding into plate assemblies, therefore, they are not perfectly flat. If these in-plane residual stresses exist in an ideally perfect plate before loading, the location of the critical points on the primary equilibrium path is modified and this can raise or lower the critical load. This is why a plate’s initial or prebuckling state is very important (i.e. nonuniformity or nonlinearity of the prebuckling configuration). Initially, plate panels are considered as having ideal and perfect behaviour. But the presence of these residual stresses makes them imperfect thereby modifying their actual behaviour. The effect of the residual stress is seen when plate initial stiffness is reduce during the yielding process. For slender plates their behaviour is asymptotic to that of the perfect plate, therefore, resulting in small reduction in strength. But for the intermediate slender plates which are common in real life application, an actual imperfect plate will have a considerably lower strength compared to the strength of a perfect plate (Stiemer, 2016). According to (Simulia, 2011), the collapse load of the actual structure is strongly affected by small changes in the direction of loads, the manner of support or changes
in geometry. This is why it becomes imperative to check whether a structure is imperfection sensitive to loads, contact, material and geometric nonlinearities. When a structure is being sensitive to imperfection, the primary path is close to a falling secondary path. As a result, prone structures contain limit loads rather than bifurcation loads. Bushnell (1981) reported that the presence of these imperfections is what reduces the maximum load carrying capacity of the plate. Note, it is not the structure that is sensitive to the imperfection but the maximum load it can safely carry.

2.4.3.4 Plate Buckling
A plate is a structural body that is initially geometrically flat, having its thickness much smaller compared to the in-plane dimensions (length and width). On the other hand, shells are structural members that are not initially geometrically flat but curved. However, unavoidable geometrical imperfections, or out-of-straightness, makes the individual plate (panel) comparable with a thin shell with small initial curvature (Dubas and Gehri, 1986). The initial geometry refers to the reference surface. When the reference surface is flat you have a plate, otherwise a shell. The reference surface is located at the mid-plane for a plate. Both plates and shells primarily carry load by membrane action. Their initial response to load action is stiff (undergoing very little deformation). But if the membrane state created by the external loading is compressive, the membrane equilibrium state will become unstable and the structure will then buckle. Given that plates are thin members, their bending response is much less stiff compared to their membrane response (Simulia, 2011; Bushnell, 1981).

Steel plates are widely used elements in buildings, bridges, automobiles, aircrafts and ships. Unlike beams and columns, which have their lengths longer than their depths and widths and so are modelled as linear members, plates have their widths comparable to their lengths so they are modelled as two-dimensional plane members. Long slender columns undergo buckling instability and steel plates under membrane compression will buckle out of their plane. The buckled shape depends on the loading and support conditions in both length and width directions. However, unlike columns, plates continue to sustain loads even after buckling and in a stable manner. Plate postbuckling capacity, especially in slender plates, can be substantially greater than the corresponding buckling strengths (critical strength) given by the expression below
\[ \sigma_{cr} = \frac{KE\pi^2}{12(1-\vartheta^2)} \left( \frac{t}{b} \right)^2 \]  

(2.38)

where \( K = \) Buckling coefficient, \( \vartheta = \) Poisson ratio, \( E = \) Young’s modulus of elasticity, \( t = \) plate thickness, \( b = \) plate depth

Plate webs, as mentioned earlier, essentially bear the shear action. When this shear action creates a compressive stress state buckling occurs. A plate buckled due to shear action, however, can still sustain additional shear effects because of its tension field action. In plate girders, web plates which are rectangular sections in nature suffer from buckling because a square element within the plate with edges oriented at 45\(^\circ\) to that of the plate experiences tensile stresses on its two opposite edges as well as compressive stresses on the remaining two edges, as shown in Figure 2.15. These compressive stresses are responsible for causing local buckling and, as a result, the plate develops waves perpendicular to the stresses. Therefore, shear buckling is basically an interaction between the destabilising diagonal compression and stabilising tension on the diagonal.

![Figure 2.15 Shear Buckling of Plates](image)

The critical shear stress at which this form of buckling occurs is defined by the equation below

\[ \tau_{cr} = \frac{KE\pi^2}{12(1-\vartheta^2)} \left( \frac{t}{b} \right)^2 \]

(2.39)

where shear buckling coefficient (\( K \)) is defined by \( K = 5.34 + 4(b/L)^2 \) when \( L \geq b \) and by \( K = 5.34(b/L)^2 + 4 \) when \( L \leq b \).
This expression is quite similar to the one for the compressive force except for the value of the buckling coefficient (K) which is different as expressed above or shown in Table (2.2) below. The buckling coefficient is a function of the aspect ratio (length/depth) and the support conditions and varies from 5.34 for an infinitely long panel to 9.34 for a square panel as presented in Table (2.2).

Table 2.2: K values for different load cases and support conditions

<table>
<thead>
<tr>
<th>Load condition</th>
<th>Support condition</th>
<th>Buckling coefficient, K</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive stress (σ)</td>
<td>Hinged-Hinged</td>
<td>4.00</td>
</tr>
<tr>
<td></td>
<td>Fixed-Fixed</td>
<td>6.97</td>
</tr>
<tr>
<td></td>
<td>Hinged-Free</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>Fixed-Free</td>
<td>0.43</td>
</tr>
<tr>
<td>Shear stress (τ)</td>
<td>Hinged-Hinged</td>
<td>5.35</td>
</tr>
<tr>
<td></td>
<td>Fixed-Fixed</td>
<td>8.99</td>
</tr>
</tbody>
</table>

It should be noted that the boundary conditions not only affect the critical buckling stress, but also influences the postbuckling behaviour. It should be mentioned that the present study is not very much concerned with postbuckling response as it aims at understanding the first point of instability of plate elements.

Given that a plate is a thin member, when its in-plane dimensions of length and width are compared to its thickness this may make it slender depending on its depth to thickness ratio (b/t). This slenderness or thinness implies that its axial (membrane) stiffness will be greater than its bending stiffness. The implication of this is that it has greater capacity to resist axial action than bending action. A thin plate can absorb a great deal of membrane strain energy without much deformation but, for an equivalent amount of bending strain energy to be absorbed, the plate must deform greatly (Bushnell, 1981). So buckling will occur when a plate
is loaded in a way that its compressive membrane strain energy is converted into bending energy. This can be occasioned by situations of eccentric loading due to corrosion. This conversion is because the plate will exchange it membrane energy for bending energy and usually, very large deformations are generally required to convert a given amount of membrane energy into bending energy. So, in environments of existence where bridges are exposed to atmospheric corrosion, it is only logical to think that plate structural components will become more slender due to corrosion degradation and so they may lose their load symmetry. More so, these environments are changing as a result of climate change and, as a result, the rate of corrosion is also likely to change due to the change in exposure conditions. Given this scenario, the buckling resistance of steel plate elements may be modified due to climate change effects, also taking into account the effects of nonuniformity or nonlinearity of boundary conditions and eccentricity of loading.

2.5 Case studies on deterioration effects on metallic bridges
On a daily basis bridges experience traffic volume and weight, deteriorating components as well as large number of stress cycles, hence the need for current condition assessment of these transport infrastructural assets (Caglayan et al., 2009). Ermopoulos and Spyrou (2006) carried out a study on condition assessment with the aim of developing a validated analytical model which will be used to assess the capacity of railway steel bridges having been degraded to carry heavier loads as specified by current standards. The key contribution of the study is the proposed scheme of strengthening for the identified deteriorated members. Calçada et al., (2002) studied the Luiz 1 Bridge in Lisbon to obtain an experimentally calibrated finite element model of the bridge under the dynamic loads applied by the new light metro of Porto. Spyrou et al. (2004) carried experimental and analytical study on historic railway bridges so as to evaluate the future traffic and axle load demands with a view to proposing strengthening and replacement regime for future upgrade as a result of degradation effects, while Imam et al. (2007) did an extensive study of the behaviour of local details such as stringer-to-floor beam connection looking at fatigue effects.

Saad-Eldeen et al., (2011) assessed experimentally the ultimate strength of severely corroded box girders subjected to uniform bending moment in midship sections in corrosive seawater environment to simulate different levels of corrosion degradation of ageing ship structures. The study found out that the load carrying capacity as well the ultimate bending moment are highly affected by corrosion deterioration of plating and the changes of material properties.
From the load-displacement and moment curvature response of the structure the identified different failure modes explaining the highly asymmetrical behavioural response of the structure. Furthermore, Saad-Eldeen et al. (2012) analysed the initial and post-collapse deflections of ship plates based on measurement records of experiments of three corroded box girders subjected to pure vertical bending loading inducing a compressive stress block on deck. The effects of initial imperfections and corrosion deterioration on the final post collapse mode shape was considered. The key finding indicated that after an inflection beta value is observed then the initial imperfection shape of the plate governs the final post collapse deformation shape. An experimental study by Saad-Eldeen et al. (2013) further evaluated the ultimate bending moment of two box girders subjected to different levels of corrosion degradation under pure vertical bending moment. One of the models was based on the degradation modelled as an average general corrosion thickness loss of several measurements while the second was modelled as the real corrosion thickness loss as measured. The main finding was that no significant difference in the ultimate capacity was observed in the case of initially corroded steel members.

Sosa et al. (2006) investigated imperfection sensitive shells to find a way of computational implementation of a lower bound approach for the buckling of such shells using general purpose FE codes and not specialised programs. The shortcoming of the later is that it limits the user from optimising solution. Results from the study show that the proposed reduced energy model can predict the lower bound load for the cylindrical shells under uniform pressure distribution.

Cruz et al. (2006) investigated the influence of web thickness reduction in shear resistance of non-prismatic tapered plate girders using data mining techniques to support finite element analysis. The effect of corrosion degradation on strength (critical and ultimate) was considered based on shear load. Findings indicated that there are different consequences on shear strength of a corroded web panel depending on the location of the corrosion. For elastic shear buckling load the central part of the panel is the most critical while for the ultimate load, thickness reduction at the left top corner is the most critical.

Boissonnade and Somja (2012) investigated numerically lateral torsional buckling phenomena in rolled and welded steel profiles. The study focused on initial imperfections particularly the influence of both geometrical and material imperfection on the bending resistance of such
members. The main findings show that adequate and reasonable realistic set of residual stresses, initial lateral imperfections and torsional twist produces consistent outcomes.

The effect of corrosion severity on ultimate strength of steel box girder in ageing ships was studied by Saad-Eldeen et al. (2013) based on experimental and numerical assessments. Findings from this study provided a new stress-strain relationship which takes into account the effect of residual stress and corrosion, it promise to be a master stress-strain curve for use in nonlinear FEA.

In other to improve on the prediction of simultaneous local and overall buckling of stiffened panels, Hughes et al. (2004) studied the ultimate behaviour of T-stiffened panels in aircrafts under uniaxial compression. The study came out with improved expressions for the elastic local plate buckling and overall panel buckling of uniaxial compressed T-stiffened sections. Equating the two expressions produced a new rigidity ratio expression useful for crossover panels (panels with both local and overall buckling stresses the same).

A study by Ozbasaran (2014) focused on the lateral torsional buckling capacity of IPN and IPE sections used as cantilever beams to support overhangs. Unlike other studies where complex close-form solutions were developed, the referred study provided a simple equation to compute lateral torsional buckling load.

Zhou et al. (2011) investigated the applicability of finite element method to collapse analysis of steel connection under compression. The study focused on the limit load estimation and found out that nonlinear buckling and modified Riks methods analyses gives a more accurate estimation of capacity. This research however, will specifically focused on the critical buckling load which is most often a nonconservative estimate of the plate strength. Hence, an assessment of that critical strength will be a very useful indices to the infrastructural manager in planning for maintenance, repair/retrofit or replacement.

Another study by Shanmugam et al. (2002) was primarily concern with finite element modelling of plate girders with web openings which in some way behaviourally can relate to pitting corrosion. Web openings in buildings and bridge construction are a solution to unacceptable large construction depths between storeys in buildings if large ducts and pipes are to be beneath beams and girders of structural steel frames, or when service space for inspection and maintenance are required in bridges.
Ahn et al. (2013) investigated the shear buckling failure modes of web panels with local corrosion numerically. The study examined corrosion damage around the supports of steel plate bridges due to humidity and rainwater deposition as well as antifreeze penetration into drainage type expansion joints. The study also considered panel’s behaviour up to the postbuckling regime. The key findings indicated shear failure mode shapes of web panels with local corrosion conforming to typical shear failures of webs with tension field band developing as a result of shear resistant behaviour of web.

Kayser et al. (1989) investigated the capacity loss due to corrosion in steel girder bridges. From the study, deterioration models were developed for the analysis and reliability evaluation of corroded steel bridges over time. Though, the study considered environmental effects on the performance of corroding steel bridges but, never envisaged and factor in or make allowances for future changing environmental challenges of climate change. However, the study acknowledges that bridges like any other structure existing in the environment deteriorates over time, therefore, proposes that modelling degraded capacity of steel bridges will require combining information about bridge location, corrosion rate of material and a suitable structural analysis method. Furthermore, the study reports that, reduction of capacity occurs faster for failure mode which depends on thin elements, particularly those in compression. Therefore, the modes of resistance that will govern the design of a new structure may not be the same for an old structure. In another study by Kayser and Nowak (1989), reckoned is made that the main causes of these deterioration in bridges superstructure are repeated live loads (fatigue) and the environment. The study reported that the effects of these deterioration condition is a reduction in both the carrying capacity of the bridge and the level of certainty concerning what this capacity will be in the future.

Chaithanya et al. (2009) studied the effects of distortion on the buckling strength of stiffened panels. Their aim was to address rational structural design procedure as they reckon that distortion whether pre-existing, fabrication-related or even initial geometrical imperfections are critical from a structural design view point. Their FE result on the effect of distortion and slenderness (thinness) on strength from the equivalent column perspective showed that the postbuckling behaviour has a significant effect, increasing strength over the predicted elastic case. However, a further FE analysis of the panel changed this conclusion. They found that stocky panels for the level of distortion investigated seem not to show the expected ultimate strength degradation from a single-column-type analysis, except for a 12mm plate-stiffener
combination which indicated significant strength degradation owing to mismatch combination. Other combinations they reported showed an increase in ultimate strength.

Das et al. (2002) look at buckling and ultimate strength criteria of stiffened shells. The aim of the study is to establish a set of design equations for buckling strength assessment for ring and stringer stiffened shells used in marine structures. Their study considered various modes of buckling under various loading conditions like axial compression, radial pressure and combined loading. They were more concern with statistical data of model uncertainty factors in term of bias and coefficient of variations to use in reliability analysis.

Moatsos and Das (2004) in their study of the effects of extreme diurnal temperature changes on ship structures, considered ultimate strength. They modelled corrosion effect on the structure using simple mathematical model based on actual measurement. Their emphasis was on the modelling of loads for use in reliability analysis. The study recognises the increasing interest from different engineering fields to include climate change effects in design.

It should be noted that most of the published work that was reviewed focused on the ultimate capacity or limit load of the structure. Asset owners and managers need to be more strategic in maintenance planning, therefore, they will require forecasting of asset deterioration over time so that they can plan their budgets accordingly given financial constraints. The information and technique to assist and support managers towards this is what this research is aimed for. This study investigates long-term environmental deterioration rates of bridge plate elements and links them with the potential influence of climate change on these.

2.6 Climate Change Background
Climate is the average and variability of certain surface weather variables like precipitation, temperature, wind and cloudiness that is experienced over a long period of time, typically in the order of 30 years or so. The averages and variability of the weather variables should be capable of statistical description and quantification (UKCIP, 2011). Weather on the other hand is the familiar hour-by-hour, day-by-day changes in temperature, cloudiness, precipitation, and other atmospheric variables (UKCIP, 2011).

Scientifically, it is the consensus that the earth’s climate is changing and these changes are believed to be caused by two major sources; natural and anthropogenic sources. Figure 2.16 shows the natural and human enhanced greenhouse effects as a result of the concentration
from each source. The natural changes are as a result of processes like oceanic circulation, plate tectonics, changes in solar radiation received by the earth, volcanic eruption and natural cycles (UKCIP, 2011). The anthropogenic changes are man dependent (human activities) like the burning of fossil fuels, deforestation, use of aerosol as coolant (Sharma, 2006). The IPCC (2013) report establishes that climate change exist with a 95% certainty that humans are responsible for it.

Figure 2.16 Natural and Human enhanced Greenhouse Effects (Goldner, 2011)

In many cases, climate change is a term often used to describe changes as a result of these anthropogenic activities rather than looking at it from the holistic perspective of any process that seeks to cause adjustments to the climate system - from a volcanic eruption to a cyclical change in solar activity (Guardian, 2010). This is so because since the mid-20th century most of the observed increases in the average global temperature are due to increase in anthropogenic greenhouse gas concentrations (IPCC 2007). Figure 2.17 show one of the ways anthropogenic greenhouse gases are released into the atmosphere. More generally, climate change is defined as “A statistical significant and lasting variation in either the mean state of the climate or its variability over an extended period, typically decades or longer, that can be attributed to either natural causes or human activity” (UKCIP, 2011).
2.6.1 Process of climate change
The earth’s main source of energy is the sun through its sunlight. Figure 2.18 shows the energy spectrum of the sun. When this sunlight hits the earth’s surface it is absorbed and the visible light (short wave radiation) is converted to heat (infrared- long wave radiation) which is radiated back into the atmosphere towards the space. Greenhouse gases in the atmosphere such as carbon dioxide, water vapour, methane absorb the infrared radiation (heat) which is converted into kinetic and potential energy. Eventually these molecules then emit heat back into the atmosphere as infrared radiation. Some of this infrared radiation is absorbed by other greenhouse gases and some absorbed at the earth’s surface and these cycles of absorption, conversion and emission are repeated. Figure 2.19 also explains this process. Essentially what this process does is to slow the loss of heat to the space, keeping the earth’s surface warmer than it would have been without the greenhouse gases (GeoscienceAustralia, 2013). Greenhouse heat trapping effects have been studied for over 150 years now by scientists and suggest that without the greenhouse effect the earth would have been cooler by about 30-35°C and life as we know would not exist (Maslin, 2009). Because these greenhouse gases are effective in keeping the planet warm, any change in their amount will affect the earth’s temperature (MetOffice, 2011).
The sixth assessment report mentions that the Earth has been in radiative imbalance with more energy from the sun entering than exiting the top of the atmosphere since at least Circa 1971-2010.

From Fig.2.19 it is observed that out of the 100% incoming solar radiation 30% of it is reflected back to space by the atmosphere, cloud and the earth surface while 70% is absorbed by the atmosphere, cloud, land and oceans giving reason to the suggested imbalance. Webster et al., 2011) points out the impact of the relative sudden rise in temperature due to CO₂
emissions to include destabilization of the earth’s weather system leading to more frequent
and severe storms, new rainfall and drought patterns and change temperature patterns leading
to cyclic temperature conditions. Key findings of the Fourth Assessment Report by IPCC
establish that warming of the climate system is unequivocal, the quantity of CO₂ in the
atmosphere in 2005 (379 ppm) exceeds by far the natural range of the last 650,000 years (180
to 300 ppm), eleven out of the twelve years period of 1995-2006 rank among the 12 warmest
years in the instrumental record since 1850, and warming in the last 100 years has caused a
global average temperature rise of about 0.74°C. The report states that annual combustion of
fossil fuel adds about 8 billion tons of carbon to the atmosphere. Many other models believe
that by 2060 annual future emissions will hit 16 billion tons (twice the current rate).
Increase in greenhouse gases in the atmosphere affects the earth’s surface energy balance.
Increase in aerosols acts to reflect and absorb incoming solar radiation and change cloud
radiative properties. It is these changes as a result of the actions of the greenhouse gases and
aerosols increases in the atmosphere that causes radiative forcing of the climate system
leading to change (IPCC, 2007). The forcing agents differ in terms of their magnitude of
forcing, spatial and temporal features. Some result in positive forcing while others to negative
forcing but both contribute to either an increase or decrease in global average surface
temperature respectively (IPCC, 2007). IPCC (2007) defines radiative forcing as “the measure
of influence a particular factor has in altering the energy balance of the earth’s atmosphere be
it incoming or outgoing energy”. This underscores the importance of the factor as a potential
climate change mechanism.
The major greenhouse gases present in the atmosphere occur naturally but their increased
concentration in the atmosphere in the last 250 years is largely due to human activities (IPCC,
2007). The Fourth Assessment Report (FAR) observed figures of atmospheric concentrations
of the different greenhouse gases are in the order of magnitude of 10⁸ and their radiative
effectiveness in the order of 10⁴. This is a reflection of their enormous diversity in properties
and origin (IPCC, 2007). Predicted concentration figures for CO₂ alone by 2080s will hit 525
parts per million (ppm) in the low emission category (an emission pathway that limits the
usage of fossil fuel) and 810 ppm in the high emission category (an emission pathway that
heavily relies on the use of fossil fuel) (Hulme et al., 2002). Comparing these figures with the
base line figures of 334 ppm for 1961-1990, an average increase of between 57% and 143% is
noticed respectively. This is almost twice or thrice the pre-industrial era concentration of 280
ppm (Hulme et al., 2002) Combining the CO₂ concentration from the Mauna Loa observatory
and that from the ice cores gives a complete record of atmospheric CO\textsubscript{2} concentration since the beginning of the industrial revolution as shown in Figure 2.20.

![Carbon Dioxide Concentrations](image)

**Figure 2.20 Carbon Dioxide Atmospheric concentration (White House Initiative on Climate Change)**

Atmospheric CO\textsubscript{2} is shown to have increased from a pre-industrial concentration of 280ppm to nearly 380ppm at present which is an increase of 100ppm (Maslin, 2009). 50ppm increase was in about 200 years while the balance 50 ppm took place in about 33 years (1973-2006). Petit *et al.* (1999) gave historical analogy of the amount of CO\textsubscript{2} in the atmosphere to range between 190 to 290 ppm in the last 200,000 years. ESRL (2011) records show rapid increase over the last 100 years to 390 ppm with an increase rate of 2 ppm/year. 14% of CO\textsubscript{2} in the atmosphere is generally believed to be as a result of burning of fossil fuel and 64% of its addition to the atmosphere since 1850 is due to fossil fuel burning (Maslin, 2009). The IPCC (2007) report declares that the scientific uncertainties of global warming are essentially resolved based on the multiple lines of independent evidences. The report states that there is clear evidence of a 0.75°C rise in global temperatures and a 22cm rise in sea level during the 20\textsuperscript{th} century. The synthesis also predicts a possible further global temperature rise of between 1.1°C and 6.4°C by 2100 and a sea level rise of between 28cm and 79cm and more if the melting of the Greenland and Antarctica accelerates. In addition, the report says that weather patterns will become less predictable and the occurrence of extreme climate events will increase. It is believed that over 60% of the average global temperature rise of 0.75°C occurred since the 1970s with nine out of ten warmest years repeating themselves during the
last 15 years with 1998 as the record warmest year globally (Nethercot, 2003). Webster et al. (2011) states that the relative sudden rise in temperature in the atmosphere is due to CO₂ emissions. CO₂ is not the most potent greenhouse gas but it is said to be the most abundant, therefore it has the largest cumulative effect. From this record of concentrations, sea level and temperature rise from independent scientist real-time observation of happenings around the world, it points to one inescapable conclusion: the climate is changing faster than anyone imagined possible even a few years ago.

### 2.6.2 Climate change impact

The built environment is certainly impacted by climate change but there have been few studies of its impact when compared to the natural environment (West and Gawith, 2005). Changes in climate variables like temperature, precipitation and wind will affect durability of buildings, bridges and their materials regardless of the cause of the climate change (Nijland et al., 2009). The effect of climate change on the durability of building material has been studied in the Netherlands with particular interest on the porous materials like bricks, natural masonry stone, concrete, timber and coatings (Steenbergen et al., 2009). Harman (2012) looked at the impact on modern building services and concluded that changes in some variables will be more important than others. For example, precipitation will have limited impact on buildings services but will have greater impact on the building exterior. Changes in wind will be very uncertain but are expected to increase in winter and decrease in summer. These changes will be significant in building and structural design. Therefore, in designing, planning and construction of buildings, account should be taken of the expected changes in climate over the structure lifespan and its services otherwise costly maintenance will be required and buildings that are uncomfortable to inhabit will be constructed.

RC has proven to be a good material in terms of structural performance and durability, however it is susceptible to environmental attack and this can severely reduce its strength and design life (Sharma and Mukherjee, 2011). This is because in humid environments, atmospheric pollutants percolate through the concrete cover and can corrode steel reinforcement. Hirsch (2010) suggested that, although some feel the risk of climate change is non-existent or insignificant because of its gradual, subtle and random effect, the impact will remain with us for the next 50 years and beyond even if the greenhouse gas emissions are stabilised today. Global average temperature has risen by about 0.75°C (Directgov, 2012) and
even a small rise such as 2°C in global temperature will have serious impacts in terms of rising sea levels, extreme events (droughts and heavy rainfall), warmer summers and wetter winters, higher winds and tidal surges on the coast (Guardian, 2010; Directgov, 2012; Evans et al., 2009). In the south-east UK, it is said that there may be a 200% increase in summer time cooling degree days by 2080s and this will have implication on building design (West and Gawith, 2005). The durability of existing structures will be subjected to heavy demand by climate change such as performing under higher temperatures and greater rainfall intensities and this can potentially increase the rate of deterioration of such structures (Hirsch, 2010; Highways Agency, 2008). High storm intensities becoming more frequent and severe are likely to render current design codes outdated for design of new structures. This means that today’s design criteria may not be able to be functionally applied in the future considering new climatic conditions of the next 50 years and beyond. Therefore, some of the existing structures at risk may require retrofitting and monitoring to ensure that their level of service, structural integrity and environmental safety is protected and maintained.

Nigeria Meteorological Agency (NIMET, 2012) reported massive floods and erosion in some parts of the country especially the coastal zones and river catchment areas in the 2012 Seasonal Rainfall Prediction (SRP). They observed that this may lead to damage of equipment, structures such as dams, roads, railways, bridges and buildings. It should be noted that a large number of houses in the country are built of materials susceptible to damage from moisture penetration. Increase in exposure to wind, rain and flood water may further compromise their structural integrity potentially leading to collapse when looking from the durability perspective. Ede (2011) observed high rate of failure of both existing and new structures in Nigeria which he suggested it can be attributable to global climate change to an extent. The study revealed that worse monthly spreads of collapses are observed in the months of March to July which corresponds to the country’s rainy season. This may imply that there is a change in the country’s rainfall pattern as a result of climate change which is now impacting on structural integrity.

This research investigates environmental deterioration of steel material and how climate change is linked and contributing to the degradation process. Rather than looking at extreme events, the purpose of this study is to focus on long-term trends and changes on climatic parameters and atmospheric pollution factors. Relevant climatic factors contributing to deterioration of steel are further reviewed and discussed in the next section.
2.6.3 Meteorological and climatic factors

2.6.3.1 Moisture
Moisture and temperature affect biological, chemical as well as mechanical processes of deterioration. The formation of a film of moisture on the material surface is dependent upon precipitation or water adsorption. This moisture film provides the medium for chemical and photochemical reaction of surface pollutants which provides the pathway for the electrochemical reaction. Two variables play a significant role in the deterioration process; dew point and relative humidity. Dew point describes the temperature at which air becomes saturated and produces dew i.e. temperature at which condensation occurs (Roberge, 2008). Dew point is a characteristic of water content of a large air mass. Relative humidity indicates the ratio of the amount of water vapour in the air at a specific temperature to the maximum amount that air could hold at that temperature expressed in percentage (Roberge, 2008). Relative humidity depends on local temperature hence a local meteorological parameter. When the temperature of a material is below the ambient dew point, water condenses on the material creating the necessary water film or layer allowing for material decay to proceed. In most materials, an increase in relative humidity allows for further deterioration due to prolonged time of wetness and better biodegradation condition (Corrosiondoctors, 2010). It should be noted that among all climatic factors, humidity plays a significant role in metal atmospheric corrosion. For metallic corrosion, the critical relative humidity value as well as the time of wetness is defined. This critical relative humidity is the minimum concentration of water vapour required for corrosion to proceed. The film of electrolyte is formed at this critical value which is also function of the corroding material.

2.6.3.2 Temperature
Temperature plays an important role in atmospheric corrosion. Temperature is said to have a gradual effect on material deterioration in a number of ways (Moncmanová, 2007). Temperature increases the rate of almost all chemical reactions like the corrosion process (Fontana & Greene, 1978). Roberge (2008) suggests that there exists a temperature lag effect on metallic objects due to their heat capacity behind changes in ambient temperature i.e. a thermal gradient between the material surface and inner layer, particularly in materials with low conductivity. This gradient which describes the change in temperature (▲T) across the boundary suggesting which direction and at what rate the temperature is changing the most results in the degradation of the mechanical properties of the material leading to cracking and
formation of anode and cathode areas. Equation 2.40 explains how the potential difference as a result of the temperature gradient is evolved. This potential difference can lead to material degradation, for example crack formation which leads to strength loss and increased material porosity which has the capacity of lowering the chemical resistance of the material. The anode and cathode areas are defined by the Nernst equation which links the equilibrium potential (E) of an electrode to its standard potential (E_o) and the concentrations or pressures of the reacting components at a given temperature. It describes the value of E for a given reactant as a function of the concentrations of all participating chemical species. This is used to determine the potential of a system. Temperature fluctuation is also believed to influence bulk expansion, dilatation of different materials in joints and expansion of water in material pores (Moncmanová, 2007). A climate with temperature fluctuations across the freezing point can cause the greatest damage, as repeated freezing cycles can destroy material surfaces.

\[ E = E_0 + 2.3 \frac{RT}{nF} \log \frac{a_{\text{oxid}}}{a_{\text{red}}} \]  

(Nernst equation) (2.40)

Where E=half-cell potential, E_o=standard half-cell potential, R=gas constant, T=absolute temperature, n=number of electron transferred, F=faraday constant, a_{\text{oxid}} & a_{\text{red}} activities (concentrations) of oxidised & reduced species.

In tropical and subtropical regions, increase in ambient temperature is a significant reason why the rate of wet deposition leads to deterioration process more than in temperate regions. Higher ambient temperature reduces the effects of freeze-thaw cycles. In polluted environments, increase in ambient temperature speeds up material deterioration because of the associated increase of chemical reaction on the surface of the material. However, reduction in ambient temperature may also promote the chance of decay. In the macro environment, increased temperature may mean increased rates of drying. It is worth mentioning that increase in temperature is a consequence of the increase in CO2 in the atmosphere.

2.6.3.3 Relative Humidity

This describes the ratio of the amount of water vapour in the air at a specific temperature to the maximum amount that the air could hold at that temperature expressed in percentage (Roberge, 2008). Humidity on the other hand is the measure of the amount of moisture in the air, i.e. it is a measure of the moisture content of the atmosphere while relative humidity is a measure of how close the air is to saturation. Among climatic factors, humidity plays an important role in atmospheric corrosion of metals. It should be noted that the absence of
atmospheric moisture will, to a large extent, hinder non pollution-induced and pollution-induced corrosion. The rate and nature of corrosion is determined by relative humidity and factors like sunlight radiation, surface pollutants, electrolyte film properties on the material and duration of effect on metal surface. Also, the rate of drying after wetting is a function of several conditions including relative humidity. For metallic corrosion, critical values of relative humidity and time of wetness should be defined because it is at this critical value that the electrolyte film necessary for the electrochemical reaction is formed. Certain products like hydrated products and hygroscopic salts can decrease the ‘critical relative humidity value’ thus resulting in high amount of moisture on the metal surface.

2.6.3.4 Wind
Wind describes air flowing from high pressure to low pressure. The effect of wind on the deterioration of materials can be viewed variedly. For example, increase in wind velocity can affect material deterioration in that it takes wind to drive liquid and solid particles from the air to the material surface where they cause local attrition and contribute to weathering of the surface. Moncmanová (2007) reported that the kinetic energy of particles and the degree of inertial impact of droplets on the material surface is a function of wind velocity. Within the wind spectrum, the high-speed end is useful when it comes to abrasion while the low-speed end is useful for diffusion. The flow of wind around a building has the capacity to influence gaseous and particulate rate of deposition as well as strengthen the effect of driving rain. Wind is said to promote wetting of walls on the windward side more than on the leeward walls during rains. It also promotes liquid solution penetration into material pores. In other weather conditions with high solar radiation and temperature changes it supports desiccation of structural parts above the ground. Both processes of wetting and drying can result in volume change effects in structures. Wind also increases the transport of sea salt inland with the potential of being able to extent the areas that will be affected by marine aerosols along seacoast (Corrosiondoctors, 2010).

2.6.3.5 Time of Wetness
This refers to the time period during which the atmospheric conditions are conducive for the formation of a moisture film on the metallic surface. It is defined as the period of time whereby relative humidity is above 80% and temperature above 0°C (Roberge, 2008). Seasonal and annual changes in rainfall affects time of wetness (TOW) i.e. time for which
surface is wet thus affecting surface leaching and moisture balance and, as a result, material degradation.

2.6.4 Air pollutants and natural atmospheric constituents
This section will consider a review of a few relevant atmospheric air pollutants that affects metals and other construction materials.

2.6.4.1 Effects of SO\textsubscript{2}
The atmosphere is a mixture of gases, liquid and solid particles. SO\textsubscript{2} is a part of the minor constituent categorised as trace gases. Among pollutants that cause damage, sulphur and nitrogen compounds and particles deserve special mention because they promote natural weathering of various materials though in some cases it may be difficult to quantify their contribution to this process. Moncmanová (2007) suggested that amongst the main gaseous pollutants that cause damage to construction material is sulphur dioxide with other pollutants like CO\textsubscript{2}, nitrogen oxides, salts spray also contributing to the degradation.

Emission of SO\textsubscript{2} from natural and anthropogenic sources is the primary source of sulphuric acid and fine sulphate particles in the air. Absorption of sulphur dioxide on porous building/constructional materials like stones causes physical changes in the material especially with regards to changes in porosity and water retention. Sulphuric aerosol has corrosive effect on most surfaces like ceilings, walls and monuments. Moncmanová (2007) suggests that deterioration of material may promote synergic effects of different types of gaseous contaminants and particles and/or their combination. Sulphur dioxide is, for instance, more reactive under higher nitrogen dioxide concentrations when increased rates of corrosion occur. This is because nitrogen dioxide oxidises sulphur dioxide to sulphur trioxide thereby promoting further absorption of sulphur dioxide and generating sulphur trioxide which subsequently reacts with moisture to form sulphuric acid.

2.6.4.2 Effects of airborne chlorides
Airborne salinity refers to the content of gaseous and suspended salts in the atmosphere (Roberge, 2008). In the case of chlorides, atmospheric pollutants such as chlorine and chlorine compounds, when present in the atmosphere, enhance atmospheric corrosion damage. They are also released into the atmosphere from both natural as well as anthropogenic sources. The corrosive effect of chlorine and hydrogen chloride tends to be stronger than those of ‘chloride salts’ anions due to the acidic character of the former.
Chloride ions tend to participate in the electrochemical corrosion reaction apart from the fact that they promote the formation of surface electrolyte by its hygroscopic (moisture absorption) action.

2.6.4.3 Effects of particles
Particles are minute granular substances that are active or inert, soluble or insoluble. These are classed into two, primary and secondary atmospheric particles. Both have principal importance in material deterioration. Some of these particles like sea-salt, wind-blown dust, volcanic emission are directly emitted while others originate from gas-phase conversions. It is believed that both emissions are approximately equal in quantity and each of these emissions can have a measured impact on the environment. Steele et al. (2003a) reported that these emissions can be linked to known environmental impacts. For example, the production of 1t of brick produces 220kg of CO$_2$ from the burning process: this has a known impact on global climate change potential. Gases can change from one form to another on cooling or by chemical reactions in the atmosphere e.g. from gas to liquid or solid particle. One issue of importance is the chemical composition and properties of these particles because it determines the deterioration capacities of these materials. The most aggressive particles are the seacoast chlorides and a variety of fly ash from municipal incinerators. Dust particles that are inert and insoluble in water practically have no impact on material deterioration except mechanical effect through abrasion of object surface. Such inert particles can also serve as carriers of aggressive chemicals or concentration site for these chemical active ions thus creating ideal conditions for oxidation process on the material surface. It is an established fact that particles are susceptible to electrostatic forces that promote deposition. Deposition of solid particles from the atmosphere can have a significant effect on metal corrosion because of the stimulating effects of these deposits leading to a reduction of the critical humidity levels through hygroscopic action and provision of anions that stimulate metal dissolution.

2.7 Summary
This chapter reviewed and discussed issues relative to steel material and climate change. The primary objective was to relate steel deterioration to environmental and atmospheric pollution parameters, which are being affected by climate change. It is obvious that every engineering infrastructure exists in an environment and this environment of existence is constantly changing due to global warming. Key questions that need investigation are: is the resistance of structural elements of assets modified under this changing environment? Can these assets
survive functionally their intended design life? What is the risk involved if they assets are deteriorating faster than anticipated? Can infrastructural managers and owners predict and strategically plan maintenance and repair budgets ahead of time? Understanding the interaction between the changing environmental conditions and structural performance is a key to providing insight to the above important questions. In the following chapters, finite element models of plate elements will be developed and analysed under different corrosion scenarios to understand the implications and, later on, this will be linked with climate change to assess the potential effects of changing environmental conditions on the buckling performance of plate elements.
CHAPTER THREE

Benchmark Finite Element Analysis and Verification Studies

3.1 Introduction
The objective of this chapter is to present a series of benchmark analyses and verification studies that will be used to validate the finite element analysis method adopted in this research work. An illustration of the application of bending and buckling theories to validate FEA results using the general purpose Abaqus commercial code software will be shown. Bending theory will be used to evaluate stress level increases in corroded steel beams subjected to atmospheric deterioration, while the buckling theory will be considered for the determination of the critical buckling load and stress in corroded plate elements. The FE model results will both be validated using close form analytical bending and buckling equations. First principle derivations will be initially demonstrated to underpin the background concepts of bending and buckling theories. Where necessary, a full nonlinear FE analysis will be run to justify the adequacy of the linear elastic outcomes or otherwise. Furthermore, a mesh sensitivity analysis will be conducted to validate mesh size, element type and number adopted, in order to gain confidence in the accuracy of the modelling techniques to be subsequently used for the analyses in the subsequent chapters.

3.2 Finite Element Characteristics
The main difference between a finite element and a rigid body is that, a finite element is deformable while a rigid body moves through space without changing shape (Simulia, 2011). While the concept of finite elements is well understood within the context of finite element program that of rigid body is somewhat new.
Abaqus as a commercial code has a variety of element types. This vast library provides a powerful set of tools for solving many problems. Basically, there are five aspect of an element that influences its behaviour (i.e. family, degree of freedom, number of nodes, formulation and integration). In Abaqus, each element has a unique identity such as T2D2, S4R, C3D8I and C3D20R. This name identifies with each of the five aspects of an element. The first letter(s) in the name tells of the family the element belong to e.g. the T in T2D2 stands for the truss element, the S in S4R stands for shell element and the C in C3D8I stands for the continuum elements. The degree of freedom defines the fundamental variables calculated during the analysis. For example, in a stress/displacement simulation the degree of freedom of relevance are the nodal translations. Some other element families such as the beam and shell have rotational degree of freedom as well. Now, in a heat transfer analysis for example, the relevant degree of freedom will be the nodal temperatures. From this, it is obvious that different types of analysis will require the use of different elements, since the degrees of freedom are not the same. However, in this research the use of elements which have translational and rotational degrees of freedom will be explore giving that attention will be giving to structural application.

Another characteristic of an element is the number of nodes (order of interpolation). Usually, these degrees of freedom (displacements, rotations, temperatures) are calculated at the nodes. At any other point in the element, these degrees of freedom are obtained by interpolation from the nodes. Normally, the interpolation order is determined by the number of nodes in the element. For elements with nodes at their corners, the use of linear interpolation in all direction is used. Therefore, they elements are referred to as linear elements or first-order elements. If the elements have midside nodes, quadratic interpolation is used, therefore, the elements are called quadratic elements or second-order elements. A modified triangle or tetrahedral elements with midside nodes also exist. These use a modified second-order interpolation, therefore, called modified elements or modified second-order elements.

For different analyses a choice of the solution technique must be made. In Abaqus, the choice is between Abaqus/standard (Implicit) or Abaqus/Explicit, see Table 3.1 for definition. Regarding the order of interpolation, Abaqus/standard offers a wide selection of both linear and quadratic elements (Simulia, 2011). Abaqus/Explicit only offers the choice of linear elements with the exception of the quadratic beam, modified tetrahedron and triangular elements. The number of nodes is typically identified in the name of an element by a number.
For example, in the brick element C3D8, the 8 represents the number of nodes, the same applies to the shell element S8R. A slight difference in convention is used for the beam family. For example, B31 represents a first-order, three-dimensional beam element while the B32 stands for a second-order, three-dimensional beam element.

Table 3.1 The major difference between an implicit and explicit analysis (Simulia, 2011)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Abaqus/standard</th>
<th>Abaqus/Explicit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solution technique</td>
<td>Uses a stiffness-based solution technique that is unconditionally stable. It is an iterative solution procedure</td>
<td>Uses an explicit integration solution technique that is conditionally stable. It is not iterative but when determining solution it advances the kinematic state from the previous increment</td>
</tr>
<tr>
<td>Disk space and memory</td>
<td>Due to the large numbers of iterations possible in an increment, disk space and memory usage can be large.</td>
<td>Disk space and memory usage is typically much smaller than that for Abaqus/standard.</td>
</tr>
</tbody>
</table>

The formulation characteristics of an element basically define the mathematical theory governing the elements behaviour e.g. the Lagrangian or material behaviour description. Another is the Eulerian or spatial description. The Lagrangian descriptor assumes that the material associated with an element remains associated with the element throughout the analysis (meaning material cannot flow across element boundaries) while the Eulerian descriptor assumes that the elements are fixed in space as the material flow through them. This is commonly used in fluid mechanics simulations.

The integration characteristics in Abaqus use numerical techniques to integrate various quantities over the volume of each element. For example, Abaqus uses Gaussian quadrature for most elements. What this means is that, Abaqus will evaluate the material response at each integration point in each element. Depending on the element, some will use full integration or
reduce integration technique a choice that has significant implication on numerical accuracy of the element for a given problem. Abaqus/standard offers both full and reduced-integration elements while Abaqus/Explicit only the reduced-integration elements, with the exception of modified tetrahedron, triangle elements and the fully integrated first-order shell and bricks elements. To recognise a reduce integration element, an R is introduced at the end of the element name to indicate that (for example, C3D20R).

In this research, the use of continuum and shell elements is explored. The choice of continuum elements is because among all the different element families they can be used to model the widest variety of structural components. Continuum elements have the flexibility of being used to build models of nearly any shape and subjected to nearly any loading (Simulia, 2011). Furthermore, this research seeks to model corrosion deterioration which has to do with section thickness loss under various patterns. The most appropriate element type to accommodate this deterioration effect is the continuum. The shell element can offer the opportunity to model uniform corrosion but when it comes to the non-uniform corrosion shapes, it will prove limited. It should be noted that, shell elements are used to model structures in which one dimension, usually the thickness, is significantly smaller than the other dimensions, therefore, the stresses in the thickness direction are considered negligible. However, a few applications were demonstrated with the shell elements for comparison.

3.3 Bending theory

3.3.1 Stresses in beams
The governing theory behind stresses in beams is the bending theory. If a beam is subjected to the action of a bending moment, there will be a variation in stresses induced through the cross-section i.e. the normal stress ($\sigma_x$) in the $x$-axis is not uniform through the section depth. The top fibre of the beam is subjected to compressive stresses while the bottom fibre to tensile stresses. These stresses depends on the magnitude of the bending moment ($M$) and the geometry of the cross-section. The normal stress or bending stress distribution is defined by

$$\frac{\sigma}{Y} = \frac{E}{R}$$

Which shows that the normal or bending stress varies linearly through the section and is a maximum at the outer fibre and zero on the neutral axis.
When the equilibrium of forces as well moments is considered about the neutral axis of the beam, the following expression is derived

\[ \frac{M}{I} = \frac{E}{R} \]  

(3.2)

Combining (3.1) and (3.2) gives:

\[ \frac{M}{I} = \frac{E}{R} = \frac{\sigma}{Y} \]  

(3.3)

Where \( M \)=Applied bending moment, \( I \)=Second moment of area about the N.A, \( \sigma \)=Bending or normal stress at distance \( Y \) from N.A, \( E \)=Young’s modulus of elasticity and \( R \)=Radius of curvature. This expression is what is commonly referred to as the simple beam theory.

Rearranging (3.3) gives:

\[ \sigma = \frac{M}{I} \rightarrow \sigma = \frac{M.Y}{I} \quad \text{or} \quad \sigma = \frac{M}{Z} \quad \text{where} \quad Z = \frac{I_{NA}}{Y_{max}} \]  

(3.4)

Z is the elastic section modulus and it is an indicator of the effectiveness of the beam section in resisting bending moment.

### 3.3.2 Application of theory

#### 3.3.2.1 Problem description

The objective of this section is to use the derived theory and validate the finite element simulation outcomes. To do this, a trial beam section of size 406×178×74kg UB (\( Z_x \)=1320cm\(^3\)) is considered for the purpose of verifying the performance of the FEM. A beam member in its as-new condition and also when it is affected by atmospheric corrosion will be simulated. One case study was adopted for the as-new beam (Figure 3.4) and two case studies for the corroded beam. From the corroded beam (Figure 3.5), the first beam cross section (Figure 3.6 & 3.7) was subjected to uniform corrosion and the second (Figure 3.8 & 3.9) to non-uniform corrosion. All these is for the purpose of observing the changes in the stress levels in the beams as a result of section thinning due to environmental deterioration. The beam is assumed to be made up of an isotropic material with young’s modulus of 205GPa and Poisson’s ratio 0.3.
Figure 3.1 Model 1(As-new beam un-deformed shape)

Figure 3.2 Typical location of corrosion on a steel girder bridge (Kayser & Norwak, 1989)
Figure 3.3 Uniform thickness loss model (Sharifi & Rahgozar, 2010)

Figure 3.4 Model 2a (uniform corrosion un-deformed shape)
Figure 3.5 Varying thickness loss model (Sharifi & Rahgozar, 2010)

Figure 3.6 Model 2b (non-uniform corrosion un-deformed shape)
3.3.2.2 Results evaluation

The FE simulation solutions are presented and summarised in Tables 3.2 and 3.3 for two load types: (a) uniformly distributed load and (b) central point load.

Comparing the FE results in Table 3.2 for element type C3D8R (linear element) for both uniform and non-uniform corroded models. It is obvious that stress levels increased. For example, the uniformly corroded model at 28% section loss stress levels rose by a 1.17 factor when compared to the as-new model. While the non-uniform corrosion scenario stress level rose by a 1.55 factor at 29% section loss. Again, from the query of the finite elements nodal displacements for both the as-new and corroded beams (i.e. uniform and non-uniform), the following maximum displacements were observed relative to the neutral axis; 8.4mm, 9.6mm and 11.2mm respectively. These values when considered from the load-deflection perspective (k=load/deflection), appear to indicate stiffness (k) is degrading with section thickness loss of which, stress level increase seem to also confirm. Therefore, it can be said that the as-new beam compared to the corroded models appear a bit stiffer. Furthermore, comparing results based on convergence p method: where convergence is attained by increasing the order of the interpolation function with changing the mesh density. Model 1 (as-new) show minimal difference between the C3D8R linear elements and the quadratic C3D20R (169.1N/mm² to 168.5N/mm²). Model 2a element refinement also show minimal difference (197.6N/mm² to 192.6N/mm²). For the as-new model element refinement, FE results draw closer to the close form solution based on the beam theory (168.5N/mm² to 152.9N/mm²), but the uniform corroded model FE results draw away from the exact solution (192.6N/mm² to 226.3N/mm²).

Table 3.3 show results for the central point load case, which also indicate increase in stress levels when comparing as-new model to the corroded model. In terms of element refinement from C3D8R to CD320R, the C3D8R elements appear to give a stiffer outcome (298.0N/mm² as compared to 306.7N/mm²), but the C3D20R element results for example, have a greater agreement to the close form solution (306.7N/mm² & 305.9N/mm²) for both the as-new model and corroded model.
Table 3.2 FEA results summary for uniformly distributed load

<table>
<thead>
<tr>
<th>Model family and type</th>
<th>Max. stress (FE) (N/mm²)</th>
<th>Theoretical stress (N/mm²)</th>
<th>% dif. In stress with respect to theoretical</th>
<th>No. of elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1 As-new</td>
<td>Solid C3D8R</td>
<td>169.1</td>
<td>152.9</td>
<td>47994</td>
</tr>
<tr>
<td></td>
<td>Solid C3D20R</td>
<td>168.5</td>
<td>10</td>
<td>47994</td>
</tr>
<tr>
<td>Model 2a Corroded uniform corrosion</td>
<td>Solid C3D8R</td>
<td>197.6</td>
<td>226.3</td>
<td>-13</td>
</tr>
<tr>
<td></td>
<td>Solid C3D20R</td>
<td>192.6</td>
<td>-15</td>
<td>33259</td>
</tr>
<tr>
<td>Model 2b non-uniform corrosion</td>
<td>C3D8R</td>
<td>262.5</td>
<td>291.2</td>
<td>-9.9</td>
</tr>
</tbody>
</table>
### Table 3.3 FEA results summary for a central point load

<table>
<thead>
<tr>
<th>Model</th>
<th>Element family and type</th>
<th>Max. stress (FE)(N/mm²)</th>
<th>Theoretical stress (N/mm²)</th>
<th>% dif. In stress with respect to theoretical</th>
<th>No. of elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model 1</td>
<td>As-new</td>
<td>Solid C3D8R</td>
<td>298.0</td>
<td>305.9</td>
<td>-3</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Solid C3D20R</td>
<td>306.7</td>
<td>305.9</td>
<td>0.3</td>
</tr>
<tr>
<td>Model 2</td>
<td>Corroded</td>
<td>Solid C3D8R</td>
<td>436.2</td>
<td>452.7</td>
<td>-4</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Solid C3D20R</td>
<td>449.7</td>
<td>452.7</td>
<td>-0.7</td>
</tr>
</tbody>
</table>

**Figure 3.7 As-new beam (deformed shape)**
Figure 3.8 Uniformly corroded beam (deformed shape)

Figure 3.9 Non-uniform corroded beam (deformed shape)
3.4 Buckling of plates

Buckling is the sudden, large, lateral deflection of a structural member due to small increase in the existing compressive axial load of the member. This compressive axial force causes the structure to deform or deflect not in the axial direction but in the lateral direction to the axial direction. When this lateral deflection occurs the structure cannot any more take in the load. Buckling can take one of the following modes; flexural buckling, lateral buckling and torsional buckling. A structural component may buckle in any one of these modes. However, only the lowest buckling value of any of the modes is of practical interest in design calculations. Examples of structures that are susceptible to buckling include columns, plates, shells and entire assemblies of structural members. Generally, buckling failure may occur by a combination of torsion and flexure and it is best addressed by a load-displacement study.

The lowest buckling load value is referred to as the critical buckling load of the structural member and can be determined using the Euler buckling expression derived as follows. Consider a rectangular thin flat plate element of length $L$, width $b$, and thickness $t$ shown in Figure (3.13) which is simply supported along all four edges. This plate is subjected to applied compressive loads ($N$) uniformly distributed over each end of the plate. When the applied loads are equal to the elastic buckling load, the plate can buckle by deflecting laterally out-of-plane by an amount $v$ into an adjacent equilibrium position defined by the expression below:

$$v = \delta \sin \frac{m \pi x}{L} \sin \frac{n \pi z}{b}$$  \hspace{1cm} (3.5)

where $m$ and $n$ indicate the number of half sine waves in the buckled mode and $\delta$ is the undetermined magnitude of the deflected shape.

The governing linear differential equation corresponding to this equilibrium position and loading is expressed as follows:

$$bD \left( \frac{\partial^4 v}{\partial x^4} + 2 \frac{\partial^4 v}{\partial x^2 \partial z^2} + \frac{\partial^4 v}{\partial z^4} \right) + N_{cr} \frac{\partial^2 v}{\partial x \partial z} = 0$$  \hspace{1cm} (3.6)

where

$$bD = \frac{Ebt^3}{12(1 - \nu^2)}$$  \hspace{1cm} (3.7)

This defines the flexural rigidity of the plate.

Equation (3.6) can be written as
\[
\frac{\partial^4 v}{\partial x^4} + 2 \frac{\partial^4 v}{\partial x^2 \partial z^2} + \frac{\partial^4 v}{\partial z^4} = \frac{12(1 - \vartheta^2)}{Et^3} \left( -N_{cr} \frac{\partial^2 v}{\partial x \partial z} \right)
\]

(3.8)

Substituting equation (3.5) into (3.8) an expression for the elastic buckling load can be defined as follows.

\[
\left( \frac{m^4 \pi^4}{L^4} + \frac{m^2 n^2 \pi^4}{L^2 b^2} + \frac{n^4 \pi^4}{b^4} \right) = \frac{12(1 - \vartheta^2)}{Et^3} \left( N_{cr} \frac{m^2 \pi^2}{L^2} \right)
\]

(3.9)

Therefore,

\[
N_{cr} = \frac{\pi^2 Et^3}{12(1 - \vartheta^2)} \left( \frac{m^2/L^2 + n^2/b^2}{m^2/L^2} \right) = \frac{\pi^2 Et^3}{12(1 - \vartheta^2)} \left( \frac{m}{L} + \frac{n^2 L}{mb^2} \right)^2
\]

(3.10)

The lowest value of the membrane buckling stress in equation (3.10) is obtained for \( n=1 \) therefore,

\[
N_{cr} = \frac{\pi^2 Et^3}{12(1 - \vartheta^2)} \left( \frac{b}{L} + \frac{1L}{mb} \right)^2
\]

(3.11)

If the quantity within the larger bracket is defined as the buckling coefficient \( k \) and noting that the buckling load \( N_{cr} \) is the product of the buckling stress, thickness (\( t \)) and width (\( b \)), the buckling stress can be expressed as

\[
\sigma_{cr} = \frac{k \pi^2 E}{12(1 - \vartheta^2)} \left( \frac{t}{b} \right)^2
\]

(3.12)

Figure 3.10 Plate buckling under uniaxial compression (Trahair et al 2008)
3.4.1 Eigenvalue plate analysis

Eigenvalue buckling analysis is generally used to compute the critical load of a stiff structure (i.e. structures that exhibit low elastic deformation). It is usually a linear perturbation solution procedure (i.e. an analysis step during which the response is linear) which most often it is the required first step before the more general load-displacement response analysis. The purpose of the eigenvalue analysis is to investigate singularities (i.e. zero stiffness) in the linear perturbation of the structure’s stiffness matrix. The aim of the eigenvalue analysis is to determine the load level at which the system equilibrium becomes unstable. This load level is the maximum load the structure can sustain. Since the eigenvalue analysis is the required first step, it therefore means, the initial condition forms the base state of the analysis; otherwise, the base state will be the current state of the model at the end of the last general analysis step. The analysis is carried out by activating the Step module in ABAQUS and then creating a step name (buckle) to run as a linear perturbation procedure type as shown in Figure 3.14.

![Figure 3.11 Abaqus Step module dialog box](image)

Abaqus/Standard allows for the use of two eigensolvers to extract the buckling modes: the Lanczos and the Subspace iteration methods. The Lanczos method is generally faster when large number of eigenmodes are required for a system with many degrees of freedom (dof), while the Subspace is faster when it comes to fewer eigenmodes usually for less than twenty as shown in Figure 3.15.
When any of the eigensolvers is invoked, a minimum and/or maximum eigenvalues of interest can be specified, and Abaqus will extract eigenvalues until either the requested number in a given range is reached or all the eigenvalues are extracted. For this research the Subspace solver was used requesting for 15 eigenvalues and allowing up to 300 maximum number of iterations. These inputs were considered sufficient because it is the lowest eigenvalue usually that is of interest and the first eigenvalue defines that. The 300 maximum iterations were inputed to allow exhaustiveness of analysis that will permit solution convergence.

3.4.2 Problem description

The objective of these benchmark analyses is to demonstrate the use of the general purpose Abaqus commercial code software in a geometric collapse study of stiff, shell-type and solid-type structures. The examples presented are those of square and rectangular, thin, elastic plates, simply supported on all four edges and compressed in one direction: uniaxial load condition.

A square plate 100mm×100mm×2mm is considered first. The plate is made up of an isotropic elastic material with a Young’s Modulus equal to 210GPa, Poisson’s ratio 0.3 and subjected to a 1N/mm shell edge load. The boundary conditions on the model are:
- Top and bottom edges: Translations U1 (x-direction), U2 (y-direction) and rotation UR1 are allowed while all other translations and rotations are constrained see Figure 3.16.

- Right and left edges: Translations U1, U2 and rotation UR2 are allowed while all other translations and rotational degree of freedom are constrained see Figure 3.17.

Figure 3.13 FE model defining Boundary condition at the top and bottom sides
The aim of this example is to enable the estimation of the buckling stress and failure modes of the plate using shell elements and identify the most appropriate modelling techniques for producing reliable results for complex situations.

3.4.3 Results evaluation

3.4.3.1 Shell-type elements

The finite element simulation returns an eigenvalue of 606.4; multiplying this with the applied shell edge load of 1N/mm gives the critical buckling load in N/mm. Since the critical buckling load is in N/mm when divided by the plate thickness gives the stress in N/mm². Therefore

$$\sigma_{cr} = \frac{N_{cr}}{t} \rightarrow \frac{606.40}{2} = 303.2 \frac{N}{mm^2}$$

To verify the FE stress value, a comparison of the FE and theory results is made thus:

$$\sigma_{cr} = K_\sigma \frac{\pi^2Et^2}{12(1-\nu^2)b^2}$$
Where $K_\sigma$ is the buckling coefficient depending on the boundary conditions. In this example since the boundary conditions are all simply supported and the aspect ratio ($m = \frac{\text{Length}(a)}{\text{Width}(b)}$) is an integer (1), $K_\sigma = 4$ therefore:

$$\sigma_{cr} = \frac{4 \cdot \pi^2 \cdot 210000 \cdot 2^2}{12(1 - 0.3^2)100^2} = 303.7 N/mm^2$$

From the above, it is observed that the percentage difference between the FE and the theoretical value is only -0.16% as presented in Table 3.4. Figure 3.18 show the first buckling mode shape for the square plate and it is consistent and in agreement with expectation and theory. Furthermore, Figure 3.19 shows the mesh convergence analysis using convergence h method: where convergence is attained by increasing mesh density without changing interpolation function, therefore, having more elements as the elements sizes are reduced. The stress outcomes do not change significantly as can be seen on the figure with different element numbers indicating reliable convergence of the results using the element type and numbers adopted.

**Table 3.4** Comparison of critical stress from FE and theory

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Theory</td>
<td>303.7N/mm²</td>
</tr>
<tr>
<td>Finite element (FE)</td>
<td>303.2N/mm²</td>
</tr>
<tr>
<td>Element type</td>
<td>S4R</td>
</tr>
<tr>
<td>Percentage difference</td>
<td>-0.16%</td>
</tr>
</tbody>
</table>
3.4.3.2 Rectangular Plate
A rectangular plate 150mm×70mm×2mm is considered having an aspect ratio increased to m=150/70=2.14. Young’s modulus of elasticity 210Gpa, Poisson ratio 0.3 and simply supported on all four edges are also assumed (similar boundary condition) to the previous square plate. The plate is analysed under a uniaxial compressive shell edge load of 1N/mm.
3.4.3.3 Results evaluation

The finite element eigenvalue for the first mode is found to be equal to 1210.6. Therefore, the critical load equals 1210.6N/mm when divided by the plate thickness of 2mm results in a critical stress of 605.3N/mm².

The theoretical critical stress can be obtained as follows:

\[ \sigma_{cr} = \frac{Kn^2Et^2}{12(1 - v^2)b^2} = 619.8 \frac{N}{mm^2} \]

The percentage difference between theory and FE is -2.3%, obtained by using element type S4R of the quadratic geometrical order as presented in Table 3.5. Note the K value (buckling coefficient) used for the theoretical computation is 4 but, when the aspect ratio of the plate is considered, which in this case is not an integer (150/70= 2.14). K will be greater than 4 thereby increasing the numerator of the stress equation thus resulting in a higher stress value; this shows that the FE result is on the conservative side. Again, Figure 3.20 show the first buckling mode shape which is consistent with expectation and theory for a rectangular plate. Figure 3.21 shows the mesh convergence analysis giving confidence to the procedural outcomes.

**Table 3.5** Comparison between FE and theoretical critical stress

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Theory</td>
<td>619.8N/mm²</td>
</tr>
<tr>
<td>Finite element</td>
<td>605.3N/mm²</td>
</tr>
<tr>
<td>Element type</td>
<td>S4R</td>
</tr>
<tr>
<td>Percentage difference</td>
<td>-2.33%</td>
</tr>
</tbody>
</table>
3.4.4 Use of Solid-type elements

Although shell elements can successfully model the buckling behavior of plates but can pose challenges when it comes to modelling corrosion and associated non-uniform thickness losses over parts of the plate. For this reason, solid elements were also investigated to determine if they would offer reliable modelling for plate buckling behavior.

For the solid elements, results agreement were achieved using different boundary conditions from those used for the shell-type elements in the previous section. The boundary conditions used on the model are as follows:
- Top and bottom edges: Translations $U_1$ (x-direction), $U_2$ (y-direction) and rotations $UR_1$ allowed free, while all other translations and rotations were constrained see Figure 3.22.
- Right and left edges: Translations $U_1$ and rotations $UR_1$ and $UR_2$ were allowed while all other translations and rotations constrained see Figure 3.23.

![Figure 3.19 FE model boundary conditions for solid elements at the top and bottom](image)

Figure 3.19 FE model boundary conditions for solid elements at the top and bottom
First, the same 150mm×70mm×2mm plate considered earlier for the buckling analysis under uniaxial load was modelled using solid elements. A Young’s Modulus of 210GPa and Poisson’s ratio of 0.3 is used. The theoretical stress is equal to:

\[
\sigma_{cr} = \frac{K\pi^2Et^2}{12(1 - v^2)b^2} = 619.76 \, N/mm^2
\]

The finite element analysis returns a 581.5 eigenvalue for the first mode which will translate into a 581.5N/mm² after multiplying with the applied 1N/mm² pressure load given that a solid element load is applied on an area. The percentage difference between the FE and theory is -6.17% as presented in Table 3.6. Figure 3.24 shows the first mode buckle shape of the rectangular plate simulated using solid elements. The deformation pattern is in line with known theory and what is expected. The mesh convergence analysis shown in Figure 3.25 indicate that the buckling load did not change much with increasing number of elements. Giving the agreement of the FE outcome to theory, it can be inferred therefore, that solid
elements can be used to model buckling behavior. More so that, the interested stress value is the critical value of which it being conservative it is an advantage.

**Table 3.6** Comparison of critical stress between FE and theory

<table>
<thead>
<tr>
<th>Theory</th>
<th>619.8N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finite element</td>
<td>581.5N/mm²</td>
</tr>
<tr>
<td>Element type</td>
<td>C3D20R</td>
</tr>
<tr>
<td>Percentage difference</td>
<td>-6.17%</td>
</tr>
</tbody>
</table>

**Figure 3.21** First mode buckle shape of a solid element plate (m= 2.14), t= 2mm
Figure 3.22 Mesh convergence for a 2.14 AR plate using solid elements

A second plate with geometric size 600mm×400mm×4mm (aspect ratio equal to 1.5) was also considered for the uniaxial load case. Similar conditions as previous are assumed, i.e. Young’s modulus of 210GPa, Poisson’s ratio of 0.3 and simply supported boundary conditions.

The Finite element analysis for this plate returns an eigenvalue equal to 79.1 which translates into a 79.1N/mm² stress giving that eigenvalue is a load dependent factor used in predicting the buckling load or stress.

Theoretically, the critical buckling stress is calculated as 75.9N/mm², therefore resulting in a 4.15% percentage difference between FE and theory as presented in Table3.7. Again, Figure 3.26 shows the first mode buckling shape for the 1.5 aspect ratio plate modelled using solid elements. As expected the failure mode shape is consistent with theory. Furthermore, Figure 3.27 shows the mesh sensitivity plot of the same plate; as indicated by the figure, the changes in the buckling stress is quite small with increase in number of elements. The observed phenomena show convergence and authenticates the reliability of the procedural outcomes. Though the FE results tend to be nonconservative in this instance though at a reasonable percentage, this situation will be taken care off when a full nonlinear analysis is performed.
Table 3.7 Comparison of critical stress between FE and theory

<table>
<thead>
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<th></th>
<th>Theory</th>
<th>Finite element</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>75.9N/mm²</td>
<td>79.1N/mm²</td>
</tr>
<tr>
<td>Element type</td>
<td>C3D20R</td>
<td></td>
</tr>
<tr>
<td>Percentage difference</td>
<td>4.15%</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.23 First mode buckle shape of solid plate (m= 1.5), t= 4mm
3.5 Discussion of results

3.5.1 Bending theory
The models described in section (3.2.2) were evaluated using Abaqus FEA results verified by the close form bending theory. The percentage difference in stress with respect to the theoretical values as presented in Table 3.2 & 3.3 were within an acceptable limit. In the case of model 2a (Table 3.2) where the percentage difference is not insignificant, the reason for this is the use of non-conforming elements C3D8R and C3D20R. These non-conforming elements follow a reduced integration technique in arriving at solution unlike the conforming elements which follow strictly the full integration rule. The advantage of using the reduced integration elements is that they result in reduced computational time though at the expense of numerical accuracy, as shown by the comparisons above. It should however be noted that, the full integration elements were not finally used here despite their potential numerical accuracy, because for these models they appear too restrictive and led to very stiff finite elements which creates convergence issues. In fact, most often the analysis aborts before achieving convergence. Non- conforming elements are widely used for plate and shell elements and their deflections are considered reasonable, even for relatively coarse meshes where the response of conforming elements appear too stiff.
3.5.2 Buckling theory
Tables 3.4 to 3.7 presented results of buckling simulations using the general purpose ABAQUS 6.14 commercial code software. The FE results indicate very good agreement with results obtained by the close form Euler buckling equation. For the plate panels using both shell elements and solid elements types, the maximum percentage difference obtained was -6.2%, resulting in conservative results from the FE results as compared to the theoretical value. The aspect ratio of the plate seems to have also played a part in the conservativeness especially for they solid elements models. From the failure mode perspective, all the models deformed consistent with the expected plate type failure mode shapes and behavior as shown in Figures 3.18, 3.20, 3.24 & 3.26. For example, Figure 3.18 shows a one half wave shape in accordance with its aspect ratio (1) and Figure 3.21 indicates a two half waves shape consistent with its aspect ratio (2.14).

3.6 Summary
The aim of the benchmark and verification chapter was to assess the suitability and appropriateness of the general purpose finite element analysis (FEA) code ABAQUS 6.14 to perform buckling analyses effectively. The key governing theories behind the behavior of the anticipated structural components, bending and buckling, have been described. These were used to validate the obtained FE results. The results showed that the ABAQUS 6.14 software has the capability to perform and deliver on the expected simulation with results within acceptable tolerance. Giving the effect of mesh size or density and number of nodes on the accuracy of results, a mesh sensitivity study of the plate buckling analyses was also carried out to determine the appropriate element type, interpolation order and number of elements that will enable solution convergence as shown in Figures 3.19, 3.21, 3.25 and 3.27 given that the main focus of this research is degradation of buckling strength. The following two chapters present and discuss results of the finite element analysis of plates of different sizes using the validated method discussed in this chapter. Chapter four will discuss buckling behaviour of plates under axial compressive loads while five will discuss the shear load scenario. The plates will be analysed in their as-new state as well as when they have suffered corrosion deterioration using different patterns and scenarios. The aim is to carry out a full mapping of the reduction in critical buckling resistance of plates under compressive and shear load situation so as to relate this to section loss due to potential environmental changes within the lifespan of an asset in Chapter six.
CHAPTER 4

The effects of corrosion on compressive buckling strength of steel plate elements

4.1 Introduction
Following the benchmark and verification studies carried out in the previous chapter to identify the most appropriate modelling techniques for buckling analysis of plates, this chapter aims at investigating the buckling strength of plate elements under different corrosion scenarios, of increasing severity. A total of 153 plates were considered for buckling analysis, both linear and non-linear, analysed under compressive loads. The results are presented in terms of both the critical buckling and ultimate strength. Buckling modes and deformed contour shapes are discussed in detail in this chapter.

4.2 Scope of the analyses
Both linear elastic eigenvalue buckling analysis as well as full static nonlinear elastic-plastic Rik’s analysis were carried out. For the compressive load case, four different corrosion patterns were simulated. The first pattern considered was the general uniform corrosion scenario where the plates suffered uniform section thickness loss over their entire area Figure 4.1.
The second pattern, which is subdivided into two different configurations 1 & 2, is the non-uniform corrosion scenario Figure 4.2 where certain zones of the plate element are affected by corrosion.

Figure 4.1 Uniform corrosion scenario

Figure 4.2 Non-uniform corrosion scenario
Each configuration is further categorised into three different corrosion conditions as follows. Condition 1 represents a scenario of web plate thickness loss at 10%, 30%, 50%, 70% and 90% while the web depth is kept corroded at 10% for each corrosion thickness percentage loss. Condition 2 is similar to Condition 1 but with 30% of the depth of the web corroded for each of the incremental thickness losses. Condition 3 is similar to the previous cases but with 50% of the depth of the web corroded for each of the incremental thickness losses. In the above scenarios, the amounts of corrosion are increased incrementally such that a spectrum of the corrosion loss and its effects on plate buckling strength can be captured and mapped. The fourth pattern represents the pit holes corrosion scenario which is modelled by nine different pit holes patterns in terms of corroded volume loss with a combination of different pit diameters. The above corrosion patterns referred to in this chapter have also been chosen to represent possible patterns observed on real life corroded plate girders for example, Figure 4.3 is representative of non-uniform corrosion configuration 2. These patterns were also idealised for ease of simulation in order to allow analysis convergence.

![Figure 4.3 Real life corrosion location and resulting pattern on a bridge](image)

The corrosion patterns were simulated using the C3D20R solid elements because of incompressibility considerations for plastic deformation in metals. The incompressible nature of plastic deformation in metals places limitation on the types of elements that can be used for
an elastic-plastic simulation (Simulia, 2011). This limitation arise because modelling material behaviour adds kinematic constraint to the element and therefore, constraining the element volume at the integration points to remain constant. For certain classes of elements, this limitation will make the element overconstrained. When these elements cannot resolve these constraints, they suffer from volumetric locking which makes their response too stiff. This locking is said to be indicated by rapid variation of hydrostatic pressure stress from element to element or integration point to integration point (Simulia, 2011). For this analysis the C3D20R solid elements were adopted because they are reduced-integration solid elements which have fewer integration points in which the incompressibility constraints must be satisfied, unlike the fully integrated, second-order, solid elements. Therefore, they C3D20R are not overconstrained hence suitable for used for this elastic-plastic simulation, as also demonstrated in the previous chapter. However, the second-order reduced-integration elements were used with caution to ensure strain requirement of 20-40% is not exceeded as it is reckoned that at this strain level they also suffer volumetric locking. Though, the effect is reducible with mesh refinement.

In chapter three, the eigenvalue analysis procedure, how it is invoke and results extracted was discussed in detail with applied examples following. It was said, the eigenvalue analysis is a perturbation procedure required as a first step before the more general load-displacement response analysis. Eigenvalue analysis outcomes are normally an overestimation of strength as the analysis is done considering that the structural component is ‘perfect’. Hence, the need for a nonlinear analysis to consider nonlinear effects of geometry and material imperfection.

4.3 Nonlinear Buckling Analysis- The Modified Riks Method
For the seeming limitation of the eigenvalue analysis where the critical buckling load of the system is estimated as a nonconservative value because of the assumption that structural components are perfect which is not so in real life. In some occasions in real life, structural components undergo finite deformations due to complex loading or material plasticity prior to buckling. Therefore, the system matrix is change which makes eigenvalue analysis inadequate to analysed the structural behaviour. Hence, the need for a nonlinear analysis. However, most nonlinear analysis tools use the Newton-Raphson solution procedure. In principle Newton-Raphson can be used to obtain solution when force decreases as a function of the displacement for a single degree of freedom problem, what is referred to as unstable
equilibrium. In the case of multi-degree of freedom problem, a stable equilibrium configuration will correspond to a local minimum energy unlike for the unstable equilibrium configuration that corresponds to a saddle point energy (least stable state). This makes the Newton-Raphson solution less likely to converge because it breaks down completely once the maximum load is reached. However, the Riks solution technique is able to determine the static equilibrium state of the model during the unstable phase of response (i.e. when the load-displacement response show negative stiffness and the structure must release strain energy to remain in equilibrium). The Riks method usually follows an eigenvalue buckling analysis in order to provide complete information about the collapse of the structure. It is found useful in speeding up convergence of ill-conditioned or snap-through problems that do not exhibit instability. Riks control analysis is one that overcomes the limitations of both the load and displacement control analyses. The load control analysis can only provide response up to the limit load giving solver limitation of matrix inversion, while the displacement control takes the response beyond the limit load to capture elasto-plastic behaviour in the direction of positive displacement. However, the displacement control analysis is limited by snap-back response but, the Riks analysis is able to provide response past these limitations using a nonphysical variable called the arc-length which enables the postbuckling and collapse behaviour to be captured. Exact postbuckling problems usually cannot be solved directly for reason of discontinuous response at the point of bifurcation or buckling. In order to analyse the problem, it must be converted into a problem with a continuous response. Normally, the conversion is done by introducing an initial imperfection into the model so that there is some response in the buckling mode before the critical load is achieved. Now, if the introduced initial imperfection is small, the deformation will also be small usually below the critical load as a result, introducing a rapid change in behaviour. This rapid transition is usually difficult to analyse. However, when the imperfection is large, the postbuckling response grows steadily before the critical load is reached. This makes the transition into the postbuckling behaviour smooth and so relatively easy to analyse. Imperfections can be introduced in a model in three ways: either as perturbation in the initial geometry of the model or by perturbation in loads or boundary conditions. In this research, imperfections where introduced as perturbations in the initial geometry of the models because the plates are most likely to behave inelastically before reaching maximum load. Such behaviour is best simulated by introducing imperfections from the linear buckling mode shapes of the eigenvalue analysis. The Riks nonlinear analysis
usually involves load application to the nodes in increments using the load proportionality factor (LPF). This enables a force versus out-of-plane displacement curve to be obtained. From the load-displacement curve, it is possible to find out for which load, P, the out-of-plane displacement (U) increases faster i.e. the structure buckles. It is reckoned that, it is at this point that the load goes from P to Pcr (the force used in estimating the buckling strength of the structure).

The Riks solution procedure is activated as follows:

1. At the *Step module in Abaqus, click on the Step manager to create a step for the analysis, defining the step name, procedure type and then click on the Static,Riks function and click the continue button to open up a dialog box to enable Edit step as shown in Figure 4.4. In the Edit step dialog box activate the nonlinear geometric function on the Basic tab.

![Figure 4.4 Step module showing the Edit Step dialog box on the Basic tab to invoke nonlinear Riks procedure](image)

2. Input sequence for the nonlinear analysis as shown in Figure 4.5 by clicking on the Incrementation tab, then activate the automatic time increment capability. Here, a
number of data line will need to be defined. The first two entries on the *Static, Riks data line to be defined are the initial and total arc lengths associated with the load in this step.

![Edit Step Incrementation tab functions](image)

**Figure 4.5 Edit Step Incrementation tab functions**

3. The next two entries to be defined are optional but forms limits for the arc length increment, these are the minimum and maximum arc length increments as shown in Figure 4.5

4. Back to the Basic tab Figure 4.4 define the last four optional entries that serves as stopping criteria for the analysis.
   - Maximum load proportionality function is provided to terminate the *Step when the load exceeds a certain magnitude.
   - The node and degree of freedom (dof) are provided to terminate the *Step when a particular displacement component exceeds a given value.

However, the analysis will not stop exactly at these values but will stop once the values are exceeded. When none of these stopping criteria is defined, the analysis will only stop when
the maximum number of increments is reached or when the solution fails because of excessive distortion for example.

It should be noted that, static problems depends on the number of iterations to achieve convergence. This underscores the importance of activating the automatic time incrementation capability to enable automatic adjustment of defined parameters during the analysis. For highly nonlinear problems, it is recommended to use a small fraction of the total step time (say 10%) as the initial increment. When the analysis is not converging it is advisable to cutback the increment size or increase the increment size when it is converging too early. For this research 1% of the total time step was found most appropriate for solution convergence and covering the required phase of the analysis. Furthermore, Abaqus aims to use 3-5 equilibrium iterations in a typical increment, but in severe cases it can allow up to 10-16 equilibrium iterations per increment.

In order to introduce the imperfection, the Edit keyword capability on the main menu bar is used as demonstrated by Figure 4.6 following the eigenvalue buckling analysis lowest buckling mode at 10% of thickness as scale factor.

![Figure 4.6 Edit Keyword for introducing imperfection](image)
A job is then created, named and submitted for analysis while the analysis progress is being monitored. After the analysis is completed based on the defined stopping criteria of 1 as maximum load proportionality factor, the results are extracted and postprocessed as follows.

1. In the Model Tree the job name is clicked to open the output database file in the visualisation module.
2. In the Result Tree the XY data is double-clicked and a create XY data dialog box comes up Figure 4.7.

![Figure 4.7 Model Result Tree to create XY data](image)

3. In the create XY data dialog box, select ODB field output and click on continue to open up the ‘XY data from ODB field output’ dialog box Figure 4.8.
4. In Figure 4.8 dialog box click on the variables tab and select Unique Nodal as output position followed by selecting U1, U2 or U3 as the case may be: spatial displacement as the output variable.

5. Click on the Elements/Nodes tab to select Node labels and then define a required node label Figure 4.9. In this research node labels were selected from the mid-plane of the plates around regions of high stress or deformation flow. Then the save button is click. This way XY data are extracted from the output using default names.
6. In the Result Tree again, double-click on XY data and an XY data dialog box comes up, select on Operate on XY data and click on the continue button Figure 4.10.
7. On the Operate on XY data dialog box, on the Operators list select on the Combine (X, X) operator so that on the text field on the top right Combine ( ) appears Figure 4.11.

![Figure 4.11 Combining XY data](image)

8. In the XY data field double-click on U1 as well as the load proportionality factor, then save the combined data object as Save As.

9. As the Save As button is clicked the save XY data dialog box appear define a name and click ok Figure 4.12.

![Figure 4.12 Defining a name on the Save XY data As dialog box](image)
10. To view the combined load-displacement plot, click on the plot expression button or return to Result Tree and click on the name U1LPF then right click, an Edit XY data dialog box will appear, and copy the data and plot curves in excel Figure 4.13.

![Edit XY Data](image)

**Figure 4.13 XY data extracted to be plotted in Excel**

After plotting the load-displacement curve, certain approaches can be utilised to estimate the critical buckling strength. For example, the linearisation method and the second derivative approach.

The linearization method which is used in this research involves creating two linear functions, one starting from the origin and following the slope of the prebuckling force path versus the out-of-plane displacement curve while the other linear function follows the postbuckling slope path, i.e. where the path shows a greater increase in displacement for small increase of the load. The intersection point between these two linear functions defines the critical buckling strength as demonstrated by Figure 4.14 (a).

The second derivative approach involves curve fitting or linearisation between points with respect to the obtained load-displacement curve from the nonlinear analysis. It is expected that an expression describing the out-of-plane displacement versus force curve will be
obtained from the curve fitting. This expression is differentiated twice with respect to the force. This way the value of the function can be calculated for all load increments. Mathematically, at the load increment where the largest value of the second derivative is obtained that corresponds to the point where the largest curvature exist on the original out-of-plane displacement versus force curve. This point defines where buckling occurs, see Figure 4.14 (b).

However, both approaches have their limitations giving that the both strongly dependent on approximations. So the quality of the nonlinear solution is critical especially for the second derivative method because of the curve fit which is highly complex to make good approximation when the structure displays an unstable postbuckling path. This is why this study adopted the linearisation approach, though it also involves a bit of arbitrary approximations depending on the defined slopes of the linear functions as well as where the second function is chosen. On the other hand, the ultimate strength of the plate was obtained at the maximum load point beyond which the stiffness of the system becomes zero.

![Diagram](image)

**Figure 4.14 (a) The linearisation method & (b) the second derivative method both use for estimating the buckling strength in a nonlinear analysis (Muameleci, 2014)**

For this study, the plate dimensions used for the analyses are representative of real bridge internal girders with geometrical characteristics of 1372mm x 686mm x 12.7mm and
The plates have aspect ratios (ARs) of 2 and 1.23, respectively, with corresponding initial slenderness ratios (b/t) of 54 and 78. Based on these slenderness ratios the plates can be classified as intermediate to slender plates given that the critical slenderness ratio (b/t) for the plate at yield stress of 275N/mm$^2$ is 52. The plate material is considered isotropic and homogeneous with a Young’s modulus of elasticity (E) of 206,000N/mm$^2$ and Poisson ratio (ν) of 0.3. The boundary conditions used are described in Table 4.1 as validated already in chapter three. The material constitutive law followed to introduce material nonlinearity in the models is presented in Figure 4.15. This study used the second stress-strain constitutive law as this is frequently used in numerical studies and generally represents the actual behaviour of structural steel in a suitable way.

<table>
<thead>
<tr>
<th>Edges</th>
<th>Translations</th>
<th>Rotations</th>
</tr>
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<td>U2</td>
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<tr>
<td>Left</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Note: 0= restraint, F=allowed free, 1, 2 & 3=x, y & z directions
4.4 Behaviour under compressive loads

As discussed earlier in chapter two, buckling failure is characterised by the sudden failure of a structural member which can take place under axial or shear loads when they create a compressive stress field in the member. This chapter considers the behaviour of the plate elements under axial load cases whereas shear load cases will be considered in the next chapter. It should be noted that when a plate element is subjected to direct compression, shear, bending, or a combination of these, the plate may buckle locally before the element as a whole becomes unstable or before the yield stress of the material is reached. This is because of the relative thinness of the plate element in comparison to its other geometrical dimensions (BCSA & SCI 2002). This local buckling phenomenon is independent of the length of the plate but dependent on a number of parameters like the aspect ratio, boundary conditions, the material yield strength, and the stress distribution across the width of the plate and residual stresses in the section (BCSA & SCI, 2002). Contrary to the view that buckling is a sudden or discontinuous phenomena, the almost inevitable presence of initial imperfections in geometry (out-of-planeness) results in the gradual growth of this cross-sectional distortion with no sudden discontinuity in the real behaviour of the member at the theoretical critical load. Hence, in this analysis an initial imperfection equal to 10% of the plate thickness is introduced in the buckling finite element analysis. It should be noted that the theoretical or elastic critical local buckling load is not on its own a satisfactory basis for design. Again, the ultimate strength of the plate may be less than the critical local buckling load due to yielding
or may be higher than the critical local buckling load due to beneficial post-buckling reserve. For example, a slender plate that is loaded in uniaxial compression with both longitudinal edges supported is expected to experience stress redistribution as well as developing transverse tensile membrane stresses after buckling so as to provide a post-buckling reserve. The initial imperfections in such a plate may cause deformations to start below the buckling load, however, the plate may still sustain loads greater than the theoretical buckling load. Despite its limitations the critical buckling load typically forms the basis for initial assessment of plates and this is important from the infrastructural management point of view and so, this research looks at both the effects of corrosion on the critical as well as the ultimate strength of plate elements. Therefore, both the general behaviour of the plate elements at the first buckling state as well the post-buckling range is considered and discussed. This is due to the fact that from an asset management point of view, initial buckling of a plate element is most likely a critical indicator required to suggest repairs or replacement to be carried out to the element to bring it to a satisfactory state. However, a designer may be interested in taking advantage of the post-buckling reserve strength which is considerably in excess of that at which the plate starts to buckle. Whatever the case, it should also be noted that the design approach of plate girder webs can be viewed from two perspectives. Firstly, the allowable stress design philosophy which is based on elastic buckling as a limiting condition and secondly, strength design which is based on ultimate strength including the post-buckling strength as a limit state (Lee et al. 1996).

4.5 Results and discussion

4.5.1 Uniform corrosion
Tables 4.2 & 4.3 presents both the critical and ultimate buckling stresses obtained from the FE analyses for the uniform corrosion scenarios for two plates with different aspect ratios (1.23 & 2). A number of different plate elements were analysed investigating the general critical and ultimate behaviour of the plate elements under compressive load action. The nonlinear FEA included both material and geometric nonlinearities. The von-Mises yield theory known to be the most suitable for ductile material such as steel, was used for the material yield criterion. Giving that out-of-plane buckling deformations do not occur in ideal flat plates under in-plane loading, a small variation from flatness of 10% of plate thickness as initial imperfection for the nonlinear analysis is introduced in order to initiate the out-of plane
deformations. The shapes of initial deformation introduced were based on the eigenvalue buckling mode shapes. The plates, as mentioned before, fall within the intermediate to slender class. The general corrosion is simulated as a progressive reduction in the thickness of the plate equal to 10%, 30%, 50%, 70% and 90% of the original thickness. The suitable mesh size for the simulation, as found by sensitivity analyses carried out in the previous chapter, is between 6000-12000 elements.

From the values in Tables 4.2 & 4.3 it is observed that, as expected, both the critical and ultimate buckling stresses decrease with plate thickness reduction due to uniform corrosion. However, in terms of the mobilisation of the post-buckling reserve strength, it seems the slender the plate the greater the mobilised reserve strength. This can be seen when the percentage difference between the FE linear critical stress and the FE ultimate stress is estimated. The decrease in stress may be due to the increase in the plate slenderness as thickness reduces, as shown in both tables. When the results of the two plates of different aspect ratios (ARs) are compared in Tables 4.2 & 4.3, it is also observed that while both plates suffer strength drop as a result of corrosion, in terms of stress magnitude the plate with the higher aspect ratio seem to have higher buckling strength values than the plate with a lower aspect ratio as shown by Figure 4.16. For example, in terms of critical strength, for a 10% reduction in plate thickness, the plate with AR equal to 2 has a critical strength of 160.7N/mm\(^2\) compared to 68.75N/mm\(^2\) for the plate with AR of 1.23; this implies a 57% increase in strength for the higher AR plate. At 90% thickness loss, the plate with AR of 2 has 69.4% higher strength as compared to the plate with AR of 1.23. Doing the same comparison for the ultimate strength results, it is observed that at 10% thickness loss, the plate with AR equal to 2 has a 35% increase in strength compared to the 1.23 AR plate while at 90% thickness loss it stands at 26% increase.
Figure 4.16 Strength comparison of plates with different aspect ratios (ARs)
Table 4.2 Compressive critical and ultimate buckling stress under uniform corrosion for aspect ratio of 1.23 & 10% initial imperfection

<table>
<thead>
<tr>
<th>Section thickness loss (%)</th>
<th>Aspect ratio (AR) 1.23</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Buckling stress (N/mm²)</td>
</tr>
<tr>
<td></td>
<td>Theoretical critical stress</td>
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<td>122.3</td>
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<td>99.1</td>
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<td>59.9</td>
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</tr>
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<td>70</td>
<td>11</td>
</tr>
<tr>
<td>90</td>
<td>1.2</td>
</tr>
</tbody>
</table>

Table 4.3 Compressive critical and ultimate buckling stress under uniform corrosion for aspect ratio of 2.0 & 10% initial imperfection

<table>
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<th>Section loss (%)</th>
<th>Aspect ratio (AR) 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Buckling stress (N/mm²)</td>
</tr>
<tr>
<td></td>
<td>Theoretical critical stress</td>
</tr>
<tr>
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</tr>
<tr>
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<td>206.8</td>
</tr>
<tr>
<td>30</td>
<td>125.1</td>
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<td>50</td>
<td>63.8</td>
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<td>70</td>
<td>22.9</td>
</tr>
<tr>
<td>90</td>
<td>2.5</td>
</tr>
</tbody>
</table>

4.5.1.1 Further verification of FEA results

In order to assess the accuracy of the FE simulation, the FE results were compared with theoretical values based on the closed form Euler buckling equation as follows.

\[
\sigma_{cr} = \frac{KE\pi^2}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2
\]
Tables 4.2 & 4.3 show that the FE critical stresses agree with excellent accuracy with the theoretical values. Furthermore, the nonlinear FE results were compared to the relevant theoretical plate ultimate strength expression as follows.

\[ \frac{\sigma_u}{\sigma_y} = \frac{2}{\beta} - \frac{1}{\beta^2} \]

where \( \beta \) defines the slenderness ratio of the plate and it is expressed as follows:

\[ \beta = \frac{b}{t} \sqrt{\frac{\sigma_y}{E}} \]

where \( b= \) plate depth, \( t= \) plate thickness, \( \sigma_y= \) material yield strength, \( E= \) young’s modulus of elasticity and \( \sigma_u= \) ultimate strength

Again, Tables 4.2 & 4.3 show that the FE ultimate strength has very good agreement with the theoretical solution.

It is also observed that, there is a reduction in the second moment of area (I) of the plates due to the corrosion degradation as can be seen in Figure 4.17.

![Figure 4.17 Relationship between second moment of area and thickness reduction](image-url)
Comparing the critical stresses from the linear and nonlinear FE analysis results Tables 4.2 & 4.3, it is observed that the linear eigenvalue results seems to be marginally non-conservative and so may not provide a reliable buckling strength generally for first step assessment of a plate element. This marginal difference is captured by Figures 4.18 & 4.19 plotted from Tables 4.4 & 4.5. The non-conservatism when computed ranges between 30% -54% for the lower AR plate and between 19% - 31% for the higher AR case.

**Table 4.4** Normalised stress values for plate with AR=1.23

<table>
<thead>
<tr>
<th>Corrosion thickness ratio thickness loss/original thickness</th>
<th>Normalised stress using as-new plate strengths AR=1.23</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LFE critical</td>
</tr>
<tr>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>0.1</td>
<td>0.81</td>
</tr>
<tr>
<td>0.3</td>
<td>0.49</td>
</tr>
<tr>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>0.7</td>
<td>0.09</td>
</tr>
<tr>
<td>0.9</td>
<td>0.009</td>
</tr>
</tbody>
</table>

**Figure 4.18** Strength reduction factors against corrosion thickness ratios for uniform corroded plate with AR=1.23
Table 4.5 Normalised stress values for plate with AR=2

<table>
<thead>
<tr>
<th>Corrosion thickness ratio thickness loss/original thickness</th>
<th>Normalised stress using as-new plate strengths AR=2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LFE critical</td>
</tr>
<tr>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>0.1</td>
<td>0.81</td>
</tr>
<tr>
<td>0.3</td>
<td>0.49</td>
</tr>
<tr>
<td>0.5</td>
<td>0.25</td>
</tr>
<tr>
<td>0.7</td>
<td>0.09</td>
</tr>
<tr>
<td>0.9</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Figure 4.19 Strength reduction factors against corrosion thickness ratios for uniform corroded plate with AR=2

Figures 4.20 and 4.21 show the load-displacement response of both plates with ARs equal to 1.23 and 2.0 obtained from the Rik’s non-linear buckling analysis, under the different amounts of uniform thickness loss scenarios. Nodal displacement of a point within area of noticeable stress distribution is extracted from the mid-plane to determine the buckling representative stress. The choice of the area with noticeable stress distribution is simply
because it shows an area in the model that first reach yield when the structure is loaded to its maximum buckling strength. The weakening effect of the thickness loss is evident as the slope of the load-displacement response progressively reduces indicating loss in the stiffness of the plate.

Figure 4.20 Load-displacement response for uniform corroded plate with AR=1.23 under uniaxial compression
From the uniform corrosion results, corrosion degrading effect on buckling strength generally begins to become critical from 30% thickness loss upwards where between 29% -99% of strength is lost for both plates (AR=1.23 & 2).

Figures 4.22 to 4.26 show the buckling mode and nonlinear deformed shapes at the ultimate stress level for both plates with ARs 1.23 and 2. The same shapes were observed when the plates suffer corrosion deterioration at the earlier stated percentages uniformly. What is obvious though, is the stress distribution on the plates at the ultimate stress level which is indicative of yielding taking place within the plates. Again, if the displacement deformed shapes are considered, for the AR=1.23 plate it is observed that a displacement of about twice the plate thickness is indicated around the middle of the plate, this could suggest loss of stiffness. For the plate with AR=2, a displacement less than the plate thickness is observed. It
should be noted that, the plate with AR=1.23 is much slender compared to the plate with AR=2 and this could be the reason for the stiffness observed.

**Figure 4.22** Eigenvalue buckling mode shape for as-new plate AR=1.23 & coordinate axes

**Figure 4.23** Stress contour at ultimate stress point for as-new plate AR=1.23
Climate Change Effects on Buckling Strength of Steel Plate Elements

Figure 4.24 Displacement contour at ultimate stress point for as-new plate AR=1.23

Figure 4.25 Eigenvalue buckling mode shape for as-new plate AR=2
Figure 4.26 Stress contour at ultimate stress point for as-new plate AR=2

Figure 4.27 Displacement contour at ultimate stress point for as-new plate AR=2

4.5.1.2 Von Mises yield criterion
A material is said to start yielding when the Von Mises stress reaches the yield strength of the material. The Von Mises stress is used to predict yielding of an isotropic and ductile material under any given complex loading condition. Results from this research were considered with respect to this yield criterion for the uniform corrosion case. The uniform corrosion model was used for the simplicity of comparison it will allow under the different corrosion thickness loss. Results can be reasonable compared since the same finite element mesh pattern can be
generated on the various corroded models and, therefore, the same nodal point can be investigated with respect to the Von-Mises stress increment.

Table 4.6 presents the Von Mises stress together with the critical and ultimate buckling stress of the plate for different amounts of corrosion. The Von Mises stress is obtained at the same nodal point for all the models located at the mid-plane within the zone of noticeable stress distribution and, this node is common to all the models giving that the models have the same mesh pattern. The results show that the Von Mises stress is increasing as expected for increasing corrosion thickness loss while the other stress components are decreasing Figure 4.22.

**Table 4.6 The effect of thickness reduction on yield strength**

<table>
<thead>
<tr>
<th>Thickness reduction</th>
<th>Nonlinear stress N/mm²</th>
<th>Ultimate stress N/mm²</th>
<th>Von-Mises stress N/mm²</th>
<th>Yield stress N/mm²</th>
<th>Yield/Von-Mises ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>122</td>
<td>158.4</td>
<td>190.1</td>
<td>275</td>
<td>1.45</td>
</tr>
<tr>
<td>0.1</td>
<td>98.9</td>
<td>143.4</td>
<td>269.9</td>
<td>275</td>
<td>1.02</td>
</tr>
<tr>
<td>0.3</td>
<td>60</td>
<td>113.2</td>
<td>303.8</td>
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<tr>
<td>0.5</td>
<td>30.7</td>
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<td>307</td>
<td>275</td>
<td>0.89</td>
</tr>
<tr>
<td>0.7</td>
<td>11.1</td>
<td>54.4</td>
<td>312</td>
<td>275</td>
<td>0.88</td>
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<tr>
<td>0.9</td>
<td>1.23</td>
<td>24.4</td>
<td>333.4</td>
<td>275</td>
<td>0.82</td>
</tr>
</tbody>
</table>
The Von Mises stress for the as-new plate at this particular node is equal to 190.1 N/mm² and this progressively rises to 333.4 N/mm², into the non-linear range, for a 90% thickness loss. A measure of the ratio of the Von Mises stress to the yield stress of the plate (275 N/mm²) is shown in Table 4.6. This ratio is confirming further how buckling strength is degrading with thickness loss. The decreasing ratio from 1.45 to 0.82 is showing how farther away from the yield stress is the Von Mises stress confirming also that the plate is most likely to fail by buckling and not yielding because of its increase slenderness as a result of corrosion thickness loss. This effect becomes critical from 30% thickness loss Figure 4.28

**4.5.2 Non-uniform corrosion**

Figures 4.29 & 4.83 show the typical non-uniform corroded plate element models simulated under uniaxial compressive load situation. These patterns are representative of corrosion patterns observed on plate girder segment between two adjacent transverse stiffeners on steel bridges. The boundary conditions on all edges of the isolated model shown in Figure 4.29 are
defined on Table 4.1. As earlier mentioned, the non-uniform corrosion scenario is subdivided into two configurations and each configuration further categorised into three conditions. A total of 120 different models were analysed to investigate the buckling response of a wide range of corroded plates. The two aspect ratios used for the uniform corrosion scenario were also adopted here for the non-uniform corroded plates.

4.5.2.1 Configuration 1 scenario
Configuration 1 is a non-uniform corrosion situation where only the bottom part of the web plate is wasted at a constant depth \( (b_c) \) of 10% (Condition 1), 30% (Condition 2) and 50% (Condition 3) while the web thickness is corroded at increasing thickness \( (t_c) \) of 10%, 30%, 50%, 70% and 90% for each of the three Conditions, as shown in Figure 4.29. Note: \( b_c \) is represented by \( \beta b \) and \( t_c \) by \( \alpha t \) on Figure 4.28.

![Figure 4.29 Non-uniform corrosion pattern-configuration 1](image)

The reason for the choice of this configuration is to represent real patterns of corrosion in bridge plate girders where the bottom part of the member often collects and holds water and
so suffers more section loss as demonstrated by Figure 4.2. Under this scenario, sixty different models were analysed and the results are shown in Tables 4.7 (for AR=1.23) & 4.8 (for AR).
Table 4.7 Compressive critical and ultimate buckling stress for non-uniform corrosion (configuration 1; 10% initial imperfection; AR=1.23)

<table>
<thead>
<tr>
<th>Thickness loss (%)</th>
<th>Aspect ratio (AR) 1.23 configuration 1</th>
<th></th>
<th></th>
<th></th>
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<tr>
<td></td>
<td>Buckling stress (N/mm²)</td>
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<td></td>
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<td></td>
<td></td>
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<tr>
<td></td>
<td>condition 1 (0.1b)</td>
<td>-</td>
<td>FE critical stress</td>
<td>FE nonlinear critical stress</td>
<td>FE ultimate stress</td>
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<td>condition 2 (0.3b)</td>
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<td>FE nonlinear critical stress</td>
</tr>
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<td>condition 3 (0.5b)</td>
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<td>FE nonlinear critical stress</td>
<td>FE ultimate stress</td>
<td>-</td>
<td>FE critical stress</td>
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Table 4.8 Compressive critical and ultimate buckling stress for non-uniform corrosion (configuration 1; 10% initial imperfection; AR=2.0)

| Thickness loss (%) | Aspect ratio (AR) 2 configuration 1 | \begin{tabular}{|c|c|c|c|c|c|c|c|c|c|}
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<tr>
<td></td>
<td>Buckling stress (N/mm²)</td>
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<td></td>
<td>condition 1 (0.1b)</td>
<td>condition 2 (0.3b)</td>
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<tr>
<td></td>
<td>FE critical stress</td>
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<td>70</td>
<td>183.5</td>
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<td>11.2</td>
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</table>

Pam Billy Fom, 2016
As expected, the plate buckling strength decreases with both thickness and depth losses as shown in Figures 4.30 & 4.31 except for 90%-30% for AR=1.23 and 10%-50% for AR=2 plates were some strength improvement is noticed for ultimate and critical stress respectively.

Figure 4.30 Buckling strength degradation with thickness-depth loss, non-uniform corrosion; configuration 1, AR=1.23

Figure 4.31 Buckling strength degradation with thickness-depth loss, non-uniform corrosion; configuration 1, AR=2
It is also evident that the difference between the results obtained from the linear critical and the non-linear critical stresses reduce as corrosion amounts increase so also the ultimate strength. For a thickness reduction factor of 0.1 and for the plate with AR equal to 1.23, the level of the depth loss in the plate (Conditions 1 to 3) does not have a significant effect on the buckling strength as does other thickness reduction factors. The percentage difference between condition 1 & 2 critical stresses is 4.8% and between condition 1 & 3 is 8.2%. For the ultimate strength scenario the percentage difference is 2.1% and 3.8% respectively. The effect of the depth starts to become more pronounced at thickness reduction factors of 0.3 and higher, were for example at 0.3 (conditions 1, 2 & 3) between 18% - 29% strength loss is observed in terms of critical stresses while ultimate strength showed between 5.7% - 9.5%. At 0.5 thickness reduction (conditions 1, 2 & 3) strength loss of between 36% - 48% is observed for critical strength while between 18%-22% is observed in terms of ultimate strength. For the higher aspect ratio plate, that is, AR=2.0 Table 4.8, the effect of the depth starts to become more significant at 0.5 thickness loss as compared to the lower aspect ratio plate. For example, at 0.5 thickness loss (conditions 1, 2, & 3) between 19% - 35% strength loss is observed for both critical and ultimate strength. For thickness reduction factors of 0.3 or less, it can be seen that there is no significant change in the buckling strength of the plate but for thickness reductions above that, the reduction in strength begins to become more pronounced.

Figures 4.32 to 4.67 show some of the buckling modes as well as the deformed model shapes or contour plots for the non-uniform corrosion configuration 1 for increasing levels of thickness and depth losses for both AR=1.23 & 2. It can be observed that for the highest amounts of thickness-depth loss, the buckling modes and deformed shapes begin to change significantly. For example, at 10%-10% thickness-depth loss, both linear buckling mode and nonlinear deformed shapes do not show any significant identifiable change from that of the as-new plate, Figure 4.22 to 4.27. However, higher thickness-depth loses such as Figures 4.46 to 4.51 for AR=1.23 and Figures 4.61 to 4.66 for AR=2 showed evidence of modification in shapes.
Figure 4.32 Eigenvalue buckling mode shape for 10%-10% thickness-depth loss
AR=1.23

Figure 4.33 Contour plot of Mises for 10%-10% thickness-depth loss at ultimate stress point AR=1.23
Figure 4.34 Displacement contour plot for 10%-10% thickness-depth loss at ultimate stress point AR=1.23

Figure 4.35 Eigenvalue buckling mode shape for 70%-10% thickness-depth loss AR=1.23
Figure 4.36 Contour plot of Mises for 70%-10% thickness-depth loss at ultimate stress point AR=1.23

Figure 4.37 Displacement contour plot for 70%-10% thickness-depth loss at ultimate stress point AR=1.23
Figure 4.38 Eigenvalue buckling mode shape for 90%-10% thickness-depth loss
AR=1.23

Figure 4.39 Contour plot of Mises for 90%-10% thickness-depth loss at ultimate stress point AR=1.23
Figure 4.40 Displacement contour plot for 90%-10% thickness-depth loss at ultimate stress point AR=1.23

Figure 4.41 Eigenvalue buckling mode shape for 10% -30% thickness-depth loss AR=1.23
Figure 4.42 Contour plot of Mises for 10%-30% thickness-depth loss at ultimate stress point AR=1.23

Figure 4.43 Displacement contour plot for 10%-30% thickness-depth loss at ultimate stress point AR=1.23
Figure 4.44 Eigenvalue buckling mode shape for 70%–30% thickness-depth loss
AR=1.23

Figure 4.45 Contour plot of Mises for 70%–30% thickness-depth loss at ultimate stress point AR=1.23
Figure 4.46 Displacement contour plot for 70%-30% thickness-depth loss at ultimate stress point AR=1.23

Figure 4.47 Eigenvalue buckling mode shape for 90%-30% thickness-depth loss AR=1.23
Figure 4.48 Contour plot of Mises for 90%-30% thickness-depth loss at ultimate stress point AR=1.23

Figure 4.49 Displacement contour plot for 90%-30% thickness-depth loss at ultimate stress point AR=1.23
Figure 4.50 Eigenvalue buckling mode shape for 90%-%50% thickness-depth loss AR=1.23

Figure 4.51 Contour plot of Mises for 90%-%50% thickness-depth loss at ultimate stress point AR=1.23
Figure 4.52 Displacement contour plot for 90%-50% thickness-depth loss at ultimate stress point AR=1.23

Figure 4.53 Eigenvalue buckling mode shape for 10%-10% thickness-depth loss AR=2

Figure 4.54 Contour plot of Mises for 10%-10% thickness-depth loss at ultimate stress point AR=2
Figure 4.55 Displacement contour plot for 10% - 10% thickness-depth loss at ultimate stress point AR=2

Figure 4.56 Eigenvalue buckling mode shape for 70% - 10% thickness-depth loss AR=2

Figure 4.57 Contour plot of Mises for 70% - 10% thickness-depth loss at ultimate stress point AR=2
Figure 4.58 Displacement contour plot for 70%-10% thickness-depth loss at ultimate stress point AR=2

Figure 4.59 Eigenvalue buckling mode shape for 90%-10% thickness-depth loss AR=2

Figure 4.60 Contour plot of Mises for 90%-10% thickness-depth loss at ultimate stress point AR=2
Figure 4.61 Displacement contour plot for 90%-10% thickness-depth loss at ultimate stress point AR=2

Figure 4.62 Eigenvalue buckling mode shape for 90%-30% thickness-depth loss AR=2

Figure 4.63 Contour plot of Mises for 90%-30% thickness-depth loss at ultimate stress point AR=2
Figure 4.64 Displacement contour plot for 90% - 30% thickness-depth loss at ultimate stress point AR=2

Figure 4.65 Eigenvalue buckling mode shape for 90% - 50% thickness-depth loss AR=2

Figure 4.66 Contour plot of Mises for 90% - 50% thickness-depth loss at ultimate stress point AR=2
Figure 4.67 Displacement contour plot for 90%-50% thickness-depth loss at ultimate stress point AR=2

For both the 10%, 30% and 50% depth loss cases Figures 4.34 to 4.39, Figures 4.43 to 4.51 AR=1.23 and Figures 4.58 to 4.66 AR=2, it can be seen that only at the very high thickness loss (70% - 90%) are they mode and contour shapes changing significantly. This means that severity of corrosion effect is determined by thickness much more than depth. It is also obvious that, there is a shift of the stress and displacement distribution on the buckling mode as well the deformed contour shapes towards the bottom of plate, this may be as a result of the change in cross section due to corrosion around that zone which is consistent with literature (Jackson and Wirtz 1983).

Again, it is observed that from the 30% depth loss cases, Figures 4.46 to 4.51 AR=1.23 and Figures 4.61 to 4.66 AR=2 that wavy deformations began to show at the corroded zone at higher thickness losses. Looking also at the Mises contour plots alongside the legend, it can be observed that with increase thickness-depth loss, the Mises stress also increases which is indicative of yielding of the plate. On the displacement contour plots, zones of the plate that have yielded show some peak displacements, subtracting the width of the peak displacements will allow the effective plate width to be determined.

Figures 4.68 and 4.68 show the load-displacement response of the plates (AR=1.23 and AR=2.0, respectively) obtained from the non-linear analysis. Generally, a drop in ultimate strength is observed but, a more noticeable drop is observed for the 70%-10% and 90%-10% thickness-depth loss patterns for both AR=1.2 and 2. In terms of stiffness, the 50% - 10%
pattern seem to exhibit higher stiffness compared to the other patterns against expectation for the AR=1.23. For the AR=2 plate, all the patterns seem to show marginal difference if not similar stiffness. Generally, stiffness variation is observed in the inelastic region. The drop in slope for the two patterns (10% -10% & 30% -10%) at the plastic region for AR=1.23 plate may be due to the effect of the out-of-plane displacement occasioned by the thinner section which may lead to plasticity of that region this is consistent with Calladine (2000) as he explains the bending moment and curvature relationship of a beam element. However, despite the drop in slope at the plastic region, the stiffness of the models appears generally the same at the elastic region.

Figure 4.68 Load-displacement response for non-uniform corroded plate configuration 1 condition 1 AR=1.23 under uniaxial compression
Figure 4.69 *Load-displacement response for non-uniform corroded plate configuration 1 condition 1 AR=2 under uniaxial compression*

Figures 4.70 to 4.75 plotted from Tables A.1 to A.6 in appendix (A), show the normalised strength plots based on the as-new strengths of the plates, for both aspect ratios, for the configuration 1 different thickness and depth loss scenarios. Considering the plate with AR of 1.23 (Figures 4.70 to 4.72), it is observed that, for the case of 10% depth loss, the buckling strength of the plate only starts reducing significantly beyond the 5% volume loss, while for the remaining cases of 30% and 50% depth loss, the buckling strength reduction is noticed even for smaller volume losses. For the case of the plate with AR=2.0 (Figure 4.73), similar trends with the AR=1.23 plate are observed. It is interesting to observe that for the 10% thickness loss case AR=2, the more severe 50% depth loss pattern resulted in higher strength than the other two patterns considering the NLFE critical stress value. This could be due to stiffness improvement caused by the way the corrosion pattern affected the plate geometry or because of the stiffness of the un-corroded upper portion of the plate. However, in terms of ultimate strength that improvement is not seen.
Figure 4.70 Reduction in strength ratios according to corrosion volume ratios, non-uniform corrosion configuration 1 condition 1 (0.1b) AR=1.23

Figure 4.71 Reduction in strength ratios according to corrosion volume ratios, non-uniform corrosion configuration 1 condition 2 (0.3b) AR=1.23
Figure 4.72 Reduction in strength ratios according to corrosion volume ratios, non-uniform corrosion configuration 1 condition 3 (0.5b) AR=1.23

Figure 4.73 Reduction in strength ratios according to corrosion volume ratios, non-uniform corrosion configuration 1 condition 1 (0.1b) AR=2
Figure 4.74 Reduction in strength ratios according to corrosion volume ratios, non-uniform corrosion configuration 1 condition 2 (0.3b) AR=2

Figure 4.75 Reduction in strength ratios according to corrosion volume ratios, non-uniform corrosion configuration 1 condition 3 (0.5b) AR=2
When the plates with different ARs but having the same depth loss are compared it is observed that they display similar behaviour, as shown Figures 4.76, 4.77 and 4.78. Again, at 10% thickness loss (Figure 4.78), the plate with AR of 1.23 shows slight improvement in buckling strength compared to the plate with AR of 2. Furthermore, Figure 4.76 shows that the plate with AR of 1.23 has higher buckling strength than that of AR of 2 for this corrosion pattern in terms of NLFE outcomes but considering the ultimate strength the AR=2 plate exhibits better strength. As mentioned above, this may be attributed to a geometric configuration development due to corrosion that seems to be contributing positively to stiffness.

![Graph showing comparison of normalized stress against Vw/Vt for different ARs](image)

**Figure 4.76** Comparison of behaviour of plates with same depth loss (0.1b) but different ARs
Figure 4.77 Comparison of behaviour of plates with same depth loss (0.3b) but different ARs

Figure 4.78 Comparison of behaviour of plates with same depth loss (0.5b) but different ARs
Figures 4.79, 4.80, 4.81 and 4.82 show the load-displacement behaviour exhibited by other corrosion patterns for configuration 1, namely conditions 2 and 3. The behaviour of both plates shows some drop in slopes reflecting stiffness degradation. However, the drop is more obvious on the AR=1.23 plate Figures 4.79 & 4.81. The behaviour displayed by the plates is generally as expected and consistent with other research. For example, Ok et al. (2007) reported that 10% corrosion loss has little effect on ultimate strength and that at higher levels of corrosion say 50% and 75% local buckling is noticed at the corroded region, which has effect on the global collapse mode of the panel.

In summary, considering the non-uniform corrosion configuration 1 scenarios, it is observed that the following patterns are the worst affected cases: 90% -10%, 70% -30%, 50% -50% thickness- depth losses where between 35% - 61% of ultimate strength is degraded for both plates (AR=1.23 & 2).

![Load-displacement response for non-uniform corrosion configuration 1 condition 2 (0.3b) under compressive load AR (1.23)](image)

**Figure 4.79 Load-displacement response for non-uniform corrosion configuration 1 condition 2 (0.3b) under compressive load AR (1.23)**
Figure 4.80 Load-displacement response for non-uniform corrosion configuration 1 condition 2(0.3b) under compressive load (AR 2)

Figure 4.81 Load-displacement response for non-uniform corrosion configuration 1 condition 3(0.5b) under compressive load (AR 1.23)
4.5.2.2 Configuration 2 scenario
In this corrosion configuration it is assumed that both the left and right edges of the plate, together with the bottom part, are wasted due to corrosion, as shown in Figure 4.83.
This corrosion pattern is representative of plate girders corrosion patterns experienced in real life, especially over supports, as can be seen in Figure 4.3. It is assumed that the left and right edges are corroded having a width of 100mm along the plate length on both sides while the bottom edge corrodes similarly to the cases investigated under configuration 1, i.e. 10%, 30% and 50% depth loss from the bottom and 10%, 30%, 50%, 70% and 90% thickness loss. The results from the analyses are presented in Tables 4.7 and 4.8 for ARs of 1.23 and 2.0, respectively. For this configuration, the differences between the linear and nonlinear results are more noticeable, as compared to configuration 1, which was discussed in the previous section. It is interesting to note from both tables that for the 10% depth loss, the 50% thickness loss results in an increase in the buckling strength of the plate as compared to the 10% thickness loss. For the 30% and 50% depth loss cases, this is not observed and buckling strength gradually decreases for increasing thickness loss. The results in Tables 4.7 and 4.8 also show that for small thickness losses (i.e. 10% and 30%), the depth of the corrosion does not have a large influence on the buckling strength of the plate. Only for higher thickness losses, above 50%, the depth starts playing a more significant role.
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<th>Thickness loss (%)</th>
<th>Aspect ratio (AR) 1.23 configuration 2</th>
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<tbody>
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<td>Buckling stress (N/mm$^2$)</td>
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<td>condition 1 (0.1b)</td>
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<td>condition 2 (0.3b)</td>
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<td>FE nonlinear critical stress</td>
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<td>FE ultimate stress</td>
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<td>FE critical stress</td>
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### Table 4.10 Compressive critical & ultimate buckling stress for non-uniform corrosion (configuration 2; 10% initial imperfection; AR=2.0)

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<tr>
<th>Thickness loss (%)</th>
<th>Aspect ratio (AR) 2 configuration 2</th>
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<tr>
<td></td>
<td>Buckling stress (N/mm²)</td>
<td>condition 1 (0.1b)</td>
<td>condition 2 (0.3b)</td>
<td>condition 3 (0.5b)</td>
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<td>42.7</td>
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<td>2.3</td>
<td>33.8</td>
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Figures 4.84 to 4.107 present the buckling modes and deformed contour shapes for a few different corrosion scenarios for configuration 2 for both AR=1.23 & 2. These shapes show the deformation evolution of the plates with increasing thickness-depth loss. In the case of linear eigenvalue analysis, the buckling mode shape changes for high thickness losses (70% and above) in all three depth loss cases. The deformed contour shapes obtained from the nonlinear analysis show local buckling occurring at the edges of the plates, which is consistent with Ok et al. (2007).

Figure 4.84 Eigenvalue buckling mode shape for 70%-10% thickness-depth loss configuration 2 AR=1.23

Figure 4.85 Contour plot of Mises for 70%-10% thickness-depth loss configuration 2 at ultimate stress point AR=1.23
Figure 4.86 Displacement contour plot for 70%–10% thickness-depth loss configuration 2 at ultimate stress point AR = 1.23

Figure 4.87 Eigenvalue buckling mode shape for 90%–10% thickness-depth loss configuration 2 AR = 1.23
Figure 4.88 Contour plot of Mises for 90%-10% thickness-depth loss configuration 2 at ultimate stress point AR=1.23

Figure 4.89 Displacement contour plot for 90%-10% thickness-depth loss configuration 2 at ultimate stress point AR=1.23
Figure 4.90 Eigenvalue buckling mode shape for 70%–30% thickness-depth loss configuration 2 AR=1.23

Figure 4.91 Contour plot of Mises for 70%–30% thickness-depth loss configuration 2 at ultimate stress point AR=1.23
Figure 4.92 Displacement contour plot for 70%-30% thickness-depth loss configuration 2 at ultimate stress point AR=1.23

Figure 4.93 Eigenvalue buckling mode shape for 70%-50% thickness-depth loss configuration 2 AR=1.23
Figure 4.94 Contour plot of Mises for 70\%-50\% thickness-depth loss configuration 2 at ultimate stress point AR=1.23

Figure 4.95 Displacement contour plot for 70\%-50\% thickness-depth loss configuration 2 at ultimate stress point AR=1.23
Figure 4.96 Eigenvalue buckling mode shape for 90%-50% thickness-depth loss configuration 2 AR=1.23

Figure 4.97 Contour plot of Mises for 90%-50% thickness-depth loss configuration 2 at ultimate stress point AR=1.23
Figure 4.98 Displacement contour plot for 90%-50% thickness-depth loss configuration 2 at ultimate stress point AR=1.23

Figure 4.99 Eigenvalue buckling mode shape for 90%-10% thickness-depth loss configuration 2 AR=2
Figure 4.100 Contour plot of Mises for 90% -10% thickness-depth loss configuration 2 at ultimate stress point AR=2

Figure 4.101 Displacement contour plot for 90% -10% thickness-depth loss configuration 2 at ultimate stress point AR=2
Figure 4.102 Eigenvalue buckling mode shape for 70%-30% thickness-depth loss configuration 2 AR=2

Figure 4.103 Contour plot of Mises for 70%-30% thickness-depth loss configuration 2 at ultimate stress point AR=2
Figure 4.104 Displacement contour plot for 70%-30% thickness-depth loss configuration 2 at ultimate stress point AR=2

Figure 4.105 Eigenvalue buckling mode shape for 90%-30% thickness-depth loss configuration 2 AR=2
Figure 4.106 Contour plot of Mises for 90%-30% thickness-depth loss configuration 2 at ultimate stress point AR=2

Figure 4.107 Displacement contour plot for 90%-30% thickness-depth loss configuration 2 at ultimate stress point AR=2

Generally, strength loss is experienced with greater section loss patterns. The most critical patterns are the 90% -10%, 70% -30% and 90%-30% thickness-depth losses for both ARs. Between 50%-85% of both critical and ultimate buckling strength is degraded due to the loss. However, as explained in previous sections, it appears some corrosion patterns seem to have ‘beneficial’ effect on buckling strength for example, the 30%-10% & 50%-10% patterns for both AR=1.23 & 2. Calladine (2000), in his book plasticity for engineers, reported that
geometry changes may work either for or against the strength of the structure or they may be neutral in effect in some circumstances. Although, he seem to refer to changes in geometry which occur when the structure deforms but the principle may be applicable here where the geometric changes is due to corrosion imperfection. This principle is viewed this way because of his further suggestion on how to apply the approach in computing the effect of geometric changes in a sequence of collapse loads. In showing further how these geometric changes can be applied, Calladine (2000) suggested that each of the collapse loads should be computed based on a geometrical configuration of a structure differing from the previous one by a small amount corresponding to the collapse mode of the previous structure. This methodology is consistent with how this research developed the models for the different corrosion loss configurations. The unfortunate thing is that, the corrosion phenomena is one that no one can control the pattern formation, therefore such seeming ‘beneficial’ patterns may not be of any engineering significance. Even though, Calladine (2000) reported that ‘beneficial’ geometry-change effects might provide an additional margin of safety when simple plastic design methods are to be used. However, he argues that ‘adverse’ geometry-change effects are the ones to be taken seriously giving that they reduce the load carrying capacity of the structure to below that which will be indicated by simple plastic theory. The simple plastic theory is premised on the assumption that the material is rigid-perfectly plastic, therefore, bounds are found on the ‘collapse’ load of the structure. This assumes also that geometry changes are unimportant.

### 4.5.3 Pitting corrosion

Pitting corrosion is a highly localised phenomenon which concentrates at one or several possible large areas (Ok et al., 2007). It occurs when local breakdown commences on a thin oxide layer on a metal surface. Once the passive layer is impacted by the attack initiated on an open surface, oxide layers form on the metal surface which reduces the rate of corrosion of the metal. When those layers are broken, they result in the accelerated dissolution of the underlying metal forming the pit surface. For this research, nine different finite element models were developed, representative of a variety of pit hole corrosion scenarios on plates, to investigate the degrading effects of pit holes on plate buckling strength and deformation. It is assumed that the pitted area should be smaller than the area of the entire dimension of the plate (FE model size) consistent with Davis (2001). The pits are of different diameters and, therefore, resulting in varying volume wastages. Pits of diameter equal to 20mm, 40mm and
50mm, at a depth of 12.7mm from the bottom of the plate, were used for the finite element analysis; these diameters were consistent with (Khedmati et al., 2011), who reported that the sizes of the pit corrosive attack can be relatively large up to about 50mm in diameter. Therefore, the pit sizes were varied in an increasing trend to study their effect on plate buckling strength. The pits were placed regularly in line with the findings of Nakai et al. (2004) that models with regularly located pits could simulate the actual corroded plate deformation where the pit diameter varies and the pits are randomly distributed.

The results from the analyses of the pit hole corrosion cases are summarised in Table 4.11.

**Table 4.11** Compressive critical and ultimate buckling stress for pitting corrosion (10% initial imperfection, AR=1.23)

<table>
<thead>
<tr>
<th>Corrosion volume ratio $V_w/V_1$ (%)</th>
<th>Aspect ratio (AR) 1.23 Pit holes</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Buckling stress (N/mm$^2$)</td>
<td>FE critical stress</td>
<td>FE nonlinear critical stress</td>
<td>FE ultimate stress</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0</td>
<td>122</td>
<td>85.7</td>
<td>158.4</td>
<td></td>
</tr>
<tr>
<td>0.26</td>
<td>121.2</td>
<td>85.7</td>
<td>155.7</td>
<td></td>
</tr>
<tr>
<td>0.52</td>
<td>120.5</td>
<td>84.3</td>
<td>150.2</td>
<td></td>
</tr>
<tr>
<td>0.78</td>
<td>120</td>
<td>84.3</td>
<td>151.9</td>
<td></td>
</tr>
<tr>
<td>0.89</td>
<td>118.8</td>
<td>82.9</td>
<td>149.7</td>
<td></td>
</tr>
<tr>
<td>2.1</td>
<td>116.2</td>
<td>82.9</td>
<td>145.7</td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>114.9</td>
<td>84.3</td>
<td>144.9</td>
<td></td>
</tr>
<tr>
<td>2.7</td>
<td>114.1</td>
<td>82.9</td>
<td>140.6</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>108.5</td>
<td>77.5</td>
<td>127.2</td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td>104</td>
<td>77.5</td>
<td>116.7</td>
<td></td>
</tr>
</tbody>
</table>
Figure 4.108 Load-displacement response of pit holes corrosion AR=1.23

It can be seen from Figure 4.108 that pitting corrosion does not seem to have a significant influence on stiffness and buckling strength of the plates, even for increasing volume wastage percentages. For example, for an increase in wasted volume by a factor of 20, the reduction in the buckling strength observed is in the order of 10 to 26% depending on the type of analysis, i.e. linear versus nonlinear. The difference in the critical buckling strength predictions between the two analysis types is in the order of 25 to 30%, the linear analysis being on the unconservative side. However, the ultimate strength is between 10% - 23% higher than the linear critical strength.

By observing the linear buckling modes and nonlinear deformed contour shapes shown in Figures 4.109 to 4.114, they seem to exhibit similar behaviour except for increase in Mises stress indicating yielding evolution due to increase in corrosion volume loss. The points of the peak out-of-plane displacements appear around the central portion of the plates. It is evident that both the buckling modes and deformed contour shapes have not been modified significantly with increasing volume loss from the normal shapes expected.
Figure 4.109 Eigenvalue buckling mode shape for 0.26% volume loss pit holes corrosion AR=1.23

Figure 4.110 Contour plot of Mises for 0.26% volume loss pit hole corrosion at ultimate stress point AR=1.23
Figure 4.111 Displacement contour plot for 0.26% volume loss pit hole corrosion at ultimate stress point AR=1.23

Figure 4.112 Eigenvalue buckling mode shape for 5.2% volume loss pit holes corrosion AR=1.23
Figure 4.113 Contour plot of Mises for 5.2% volume loss pit hole corrosion at ultimate stress point AR=1.23

Figure 4.114 Displacement contour plot for 5.2% volume loss pit hole corrosion at ultimate stress point AR=1.23
4.5.4 Equivalent corrosion cases

The previous sections presented the results from the buckling analysis of plates under different corrosion scenarios and patterns. For the purpose of obtaining a better insight towards the criticality of the different patterns, a number of analysis have been carried out assuming the same material volume loss (wastage) but under different corrosion patterns. Tables 4.12, 4.13 and 4.14 present results of these equivalent corrosion cases covering uniform corrosion, non-uniform corrosion and pit hole corrosion scenarios, respectively, all under the same amount of wastage. Based on the severity and intensity of the corrosion case, the material loss was converted into an overall reduction in plate thickness, depth or pit hole diameters.

Table 4.12 Non-uniform corrosion configuration 1 cases equivalent to 10% uniform thickness loss (compressive load case, imperfection scale factor (10%))

<table>
<thead>
<tr>
<th>Model</th>
<th>b_c (mm)</th>
<th>t_c nonunif (mm)</th>
<th>Equivalent volume (mm^3)</th>
<th>Aspect ratio (1.23), length (1216mm)</th>
<th>Buckling stress (N/mm^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>LFE crit.</td>
<td>NLFE crit.</td>
</tr>
<tr>
<td>Model 1</td>
<td>198.2</td>
<td>6.35</td>
<td>1,530,421</td>
<td>41.3</td>
<td>27.5</td>
</tr>
<tr>
<td>Model 2</td>
<td>297.3</td>
<td>4.23</td>
<td>1,530,421</td>
<td>66.2</td>
<td>45.0</td>
</tr>
<tr>
<td>Model 3</td>
<td>495.5</td>
<td>2.54</td>
<td>1,530,421</td>
<td>87.7</td>
<td>61.3</td>
</tr>
</tbody>
</table>
Table 4.13 Various non-uniform corrosion configuration 1 cases equivalent of 10% - 95% uniform corrosion (compressive load case, imperfection scale factor (10%))

<table>
<thead>
<tr>
<th>Thickness reduction factor in uniform corrosion</th>
<th>Equivalent volume loss in non-uniform corrosion (mm³)</th>
<th>bₜ (mm)</th>
<th>tₜ (mm)</th>
<th>Buckling stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>1,530,421</td>
<td>198.2</td>
<td>6.35</td>
<td>41.3</td>
</tr>
<tr>
<td>0.3</td>
<td>4,591,263</td>
<td>495.5</td>
<td>7.62</td>
<td>46.8</td>
</tr>
<tr>
<td>0.5</td>
<td>7,652,106</td>
<td>693.7</td>
<td>9.07</td>
<td>25.3</td>
</tr>
<tr>
<td>0.7</td>
<td>10,712,948</td>
<td>792.8</td>
<td>11.1</td>
<td>3.67</td>
</tr>
<tr>
<td>0.95</td>
<td>13,773,790</td>
<td>941.5</td>
<td>12.0</td>
<td>0.51</td>
</tr>
</tbody>
</table>

Table 4.14 Pit holes equivalents of uniform corrosion compressive load case

<table>
<thead>
<tr>
<th>Thickness reduction factor in uniform corrosion</th>
<th>Equivalent pit volume loss (mm³)</th>
<th>Radius of pit r (mm)</th>
<th>Pit depth h (mm)</th>
<th>Critical buckling stress (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>1,530,421</td>
<td>195.8</td>
<td>12.7</td>
<td>105.1</td>
</tr>
<tr>
<td>0.3</td>
<td>4,591,263</td>
<td>339.2</td>
<td>12.7</td>
<td>69.4</td>
</tr>
</tbody>
</table>

Table 4.12 presents the case of a 10% uniform corrosion loss converted to an equivalent volume waste in the case of non-uniform corrosion configuration 1 represented by a thickness and depth loss equivalent that will give the same volume loss with the 10% uniform case. Three different cases were analysed for different combinations of thickness and depth loss. For each of the thickness and depth combination the critical buckling stress was evaluated through linear and nonlinear analysis. From the results presented in Table 4.12, it
can be seen that the thickness-depth combination with the greater thickness component results in the lowest buckling strength (critical & ultimate) in the plate as opposed to the case of greater depth reduction, meaning thickness reduction is most critical to strength reduction. Again, for the uniform corrosion at 10% thickness loss a 68.8N/mm² and 143.4N/mm² nonlinear critical and ultimate strengths are obtained (Table 4.2) as strength against 61.3N/mm², 45N/mm² 27.5N/mm² (critical) and 135.6N/mm², 117.4N/mm² and 93.6N/mm² (ultimate) were obtained for the equivalent 10% (thickness-depth) combinations in non-uniform corrosion (Table 4.12). This means that the non-uniform corrosion pattern is a worst case. Table 4.12 presents further cases of non-uniform corrosion, of increasing wastage volume, equivalent to uniform cases. The results generally show decrease in the critical and ultimate buckling strength with increase in plate thickness and depth loss. Comparing the results in Tables 4.2 & 4.12 indicate that the equivalent non-uniform cases degrade strength more. Table 4.13 presents results of the pit holes corrosion equivalents of the uniform corrosion. The results indicate that greater pit radius degrade plate strength more.

4.6 Concluding remarks

Corrosion is material degradation resulting from material interaction with its environment (Fontana, 2013; Nakai et al., 2006). Metal corrosion is an electrochemical process that is likely to take place in the environment of existence of an asset, like a metallic bridge, unless the metal is well protected by techniques like coating or sacrificial anodes. Traditionally, the concept of ‘corrosion margin’ and ‘allowable corrosion level’ have being used by designers to protect the asset against corrosion. For corrosion margin an additional thickness tolerance is provided at the time of design while for the allowable corrosion level a guidance to determine when to renew a deteriorated member at the time of maintenance is provided (Nakai et al., 2006). Coating involves painting of the metal surface to prevent the electrolytes from reaching the metal surface, while sacrificial anode involves attaching a metal more anodic than the metal to be protected thereby forcing the structural metal to be cathodic thus sparing it from corrosion. On the whole these methods are common and generally effective. However, in complex changing environmental conditions, affected by climate change, influenced by the interaction between pollutants, temperature, relative humidity and the time of wetness, assessing the impacts of these on the deterioration amounts and rates is quite challenging. This is where the current research becomes significant because it leads into a better understanding of the long-term effects of corrosion on the buckling strength of plates. As a
result, it can enable the application of load controls, the prediction of potential failure and/or remaining service life, and also defining relevant inspection and maintenance regimes when traditional corrosion protection is being challenged by changing environmental condition.

In this chapter, results from the FE analysis of plates under different corrosion scenarios and patterns have been presented. The plates analysed had geometric dimensions picked from real elements of internal girders of a bridge. The plates had aspect ratios of 1.23 and 2 with slenderness ratios of 54 and 78 which categorised them as slender and intermediate with respect to buckling behaviour. The plates were subjected to compressive loads in order to determine the degrading effects of corrosion on their buckling strength and deformation characteristics. Both linear and nonlinear analyses were carried out. The static, Riks full nonlinear analysis technique was employed in simulating the plates. The method ensured that the static equilibrium state of the models during the unstable phase of response was determined. Again, the method follows an eigenvalue buckling analysis to provide complete information about the plates collapse. The Riks algorithm was chosen because it speeds up convergence of ill-conditioned or snap-through problems that do not exhibit instability. At least four corrosion patterns representative of real corrosion scenarios were simulated in an idealised manner: uniform corrosion; non-uniform corrosion in two configurations of three different conditions and pit holes corrosion were investigated. Generally, it appears from the presented results that corrosion has a fundamental compromising effect on structural mechanical response which may in turn degrade structural resistance to buckling. For example, increase corrosion deterioration lead to strength reduction, modification of deformation characteristics like buckling modes and deformed contour shapes, increase slenderness and reduction in geometric stiffness (I). The next chapter will look at the behaviour of plate elements under shear load.
CHAPTER 5

The effects of corrosion deterioration on shear buckling strength of steel plates

5.1 Introduction
In this chapter, numerical FE results of simulations carried on corroded steel plate elements under shear load are presented. The finite element modelling of uniform and non-uniform corrosion configurations observed on metallic bridges exposed to atmospheric corrosion as used in the axial compressive load case in chapter four are considered. The same uniform and non-uniform corrosion patterns were used for performance assessment of the shear buckling strength of these plates. The boundary conditions (BC) used to simulate the models under pure shear loading are described in Table 5.1. Initial geometric imperfections will be applied at 10% scale factor of thickness following the first eigenmode for all models, this is to initiate the buckling phenomena. The aim in this chapter is to carry out a full performance mapping of corroded steel plate elements under shear loading. The FE outcomes will then be compared with theoretical analytical solutions in other to validate the procedure. Summarising curves of strength reduction versus section loss will be produced as a key objective of this research.

5.1.1 Basic mechanism of shear
Rectangular plates loaded in shear such as web plates in plate girders are susceptible to buckling. The behaviour of a plate in shear is different to that under compression. The response of an isolated plate can be separated into two distinct phases, that is, prebuckling phase and the tension field phase Dubas and Gehri (1986). Prior to buckling the stress is
essentially a combination of diagonal tensile and compressive components of equal magnitude. The compressive principal stress essentially causes the destabilising effects that result in buckling of the panel and because of this dual stress system the buckling coefficient $K$ is somewhat higher than for the case of compression load Dubas and Gehri (1986). For a web element with a large depth to thickness ratio ($b/t$) its shear capacity is governed by elastic shear buckling, therefore, its elastic critical shear buckling stress can be calculated using the Euler buckling equation. However, the relevant elastic shear buckling coefficient $K$ should be determined and this is dependent on the boundary condition at the juncture between the web and the flange elements of the plate girder. The real boundary condition at the juncture is said to be somewhere between simple and fixed condition and this has been recognised since early days by researchers. For example, in the case of plate girders, Basler (1963) and Porter et al. (1987) assumed that the web panel is simply supported at the juncture but, Chern and Ostapenko (1969) assumed that the juncture is somewhat fixed. While Sharp and Clark (1971) assumed intuitively that the flange to web boundary condition is half way between simply supported and fixed conditions for plate girders. Research by Lee et al. (1996) show that the boundary condition at the flange-web juncture is much closer to fixity for plate girders. They show that the assumption that the web panel is simply supported at the juncture leads to a considerable underestimation of the ultimate shear strength because of the underestimation of the elastic shear buckling strength of the plate girder. Therefore, they proposed simple equations for the determination of the shear buckling coefficients for plate girder web panel in terms of web plates with simple-simple and simple-fixed boundary conditions. Details of these equations can be found in their paper.

The second phase is that which once the critical buckling has occurred the behaviour of the plate is again stable in the elastic regime but this time because the loading is resisted by an increase in the diagonal tensile force component. However, this is only possible when the panel is surrounded by stiffening members. For example, if the panel is part of a girder structure the flanges themselves may well contribute to the shear carrying strength by way of a Vierendeel action. This takes place essentially once the critical buckling and tension field actions have been exhausted.
5.2 Shear load buckling strength evaluation

5.2.1 Boundary conditions
For the shear load case, the following boundary conditions (BC) were used for the three dimensional models, see Tables 5.1. At the top and bottom edges of the plate the x-axis translation (U1) was restrained while they y and z axes translations U2, U3 and the rotations, UR1, UR2 and UR3 were allowed to move freely. For the right and left edges translations U1 were both restrained from movement as well as U2 and U3 for the left edge particularly. In terms of rotation, they rotations UR1, UR2 and UR3 for both edges were allowed to be free. These boundary conditions (BC’s) produced the expected linear buckling mode shapes as can be seen in Figures 5.1.

![Buckling mode shapes](image)

Figure 5.1 Buckling mode shapes for plates with AR 1.23 & 2 respectively under uniform corrosion showing also the coordinate axes

Again, there is good agreement between the theoretical and the FE outcomes which lies between 6% & 7%, see computation in section 5.2.2.
Table 5.1 Boundary conditions for the three dimensional models using solid elements

<table>
<thead>
<tr>
<th>Edges</th>
<th>Translations</th>
<th>Rotations</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UI</td>
<td>U2</td>
</tr>
<tr>
<td>Top</td>
<td>0</td>
<td>F</td>
</tr>
<tr>
<td>Bottom</td>
<td>0</td>
<td>F</td>
</tr>
<tr>
<td>Right</td>
<td>0</td>
<td>F</td>
</tr>
<tr>
<td>Left</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Note: 0= restraint, F= free

5.2.2 Theoretical critical strength computation

\[
\tau_{cr} = \frac{KE\pi^2}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2
\]  

(5.1)

Where \(E=206000\text{N/mm}^2\), \(\nu=0.3\) and \(K\) (buckling coefficient) for simply supported condition is defined by \(K=5.34 +4(d/L)^2\) for \(L \geq d\) or \(K=5.34(d/l)^2 + 4\) when \(L \leq d\)

For plate of geometrical dimension \(1216\text{mm} \times 991\text{mm} \times 12.7\text{mm}\) (AR=1.23), \(K\) is computed to be 7.99 giving that \(L \geq d\), therefore, theoretical shear stress is:

\[
\tau_{cr} = 244.38\text{N/mm}^2
\]

FE value; linear: 259.3N/mm² from Table 5.2 which is 6% higher than the theoretical value.

For the plate with geometrical dimension \(1372\text{mm} \times 686\text{mm} \times 12.7\text{mm}\) (AR=2), \(K = 6.34\) therefore, theoretical shear stress is:

\[
\tau_{cr} = 404.67\text{N/mm}^2
\]

FE value; linear: 418.73N/mm² from Table 5.3 which is 3.4% more than theoretical value.

From the above computations, it is clear that, there is good agreement between FE and theoretical results. Again, they two verification observations of buckling mode shapes and elastic critical shear stress further validates the adopted FE procedure.
5.3 Uniform corrosion scenario

Tables 5.2 and 5.3 presents shear buckling stress results under uniform corrosion scenario from the relevant boundary conditions (BC’s) as defined in Table 5.1 for plates with AR 1.23 & 2. From the results it can be said that the FE results are a bit nonconservative when viewed in comparison with the theoretical outcomes. Again, it is observed that the ultimate stress at lower depth to thickness ratio (b/t) is not affected significantly by the post critical strength as compared to when the slenderness (b/t) ratio increases. The post critical effect is observed to begin to set in from the 50% thickness loss situations where slenderness is seen to be increasing. This seem to agree with standard literature which explain that for stocky unstiffened web plate in pure shear, the web behaves elastically in shear until first yield which occurs at $\tau_y = \frac{f_y}{\sqrt{3}}$ and then undergoes increasing plasticity until the web is fully yielded in shear (Trahair et.al, 2008; Dubas & Gehri, 1986). Literature further explain that, because the shear stress distribution at first yield is almost uniform, the nominal first yield and the fully plastic loads are nearly equal with shear shape factor close to unity. This is why stocky unstiffened plates reach first yield before they buckle elastically, and so their resistances are determined by the shear stress $\tau_y$. Again, it is observed that while plate slenderness ratio increases the second moment of area decreases indicating degradation of geometrical stiffness. Comparing Tables 5.2 and 5.3 giving that they Tables presents results of plates with different aspect ratios, it is observed that the plate with higher aspect ratio seems to have higher strength compared to the plate with a lower aspect ratio in terms of both critical linear elastic and ultimate strength results, though it may be considered marginal difference with respect to ultimate stress.

As earlier mentioned, a full nonlinear FEA was carried to obtain the ultimate stress of the plate element. It should be noted that, the general sources of nonlinearity includes geometry, material and boundary conditions. Initial geometrical imperfections are reported to cause growth of out-of-plane deformations from the onset of loading Dubas and Gehri (1986). The effect of these deformations on collapse, however, is less obvious in some instances because of the presence of the tensile element of load resistance. This point is corroborated by Featherston (2001) who reported that the effect of initial imperfections is minimal on the postbuckling behaviour. Furthermore, the imperfect model does not exhibit the same clear cut separation between prebuckling phase and tension field phase. For slender plate panels the deformation grows progressively and the ratio of diagonal tensile to compressive stresses
gradually increases Dubas and Gehri (1986). This may be the reason for the contribution of post critical strength observed beyond 30% thickness loss. The tensile component, being imperfection insensitive, dominates the behaviour to some extent and the overall effect is to reduce the sensitivity of shear panel strength to imperfection level. The very conservative outcome of the nonlinear results and how they quite logically relate with the material first yield value $\tau_y$, presents a reasonable good degree of confidence in the nonlinear FEA structural modelling technique. Again, carrying out verification on the FE results using the theoretical close form solution confirms the appropriateness of the modelling technique. For example, at 10% thickness loss 90% of the thickness will remain for uniform corrosion. From the close form equation critical shear stress is directly proportional to thickness square. Therefore, substituting for thickness as 90% (0.9) square and multiplying with the as-new linear strength from Table 5.2 gives 210.03N/mm² compared to 210.27N/mm² obtained from FE at 10% thickness reduction. Checking for 30%, 127.06N/mm² is obtained compared to 127.43N/mm² from FE. This shows that the FE results are 6% higher than theory.

**Table 5.2** Critical & ultimate buckling stresses for uniform corrosion at 10% initial imperfection shear load case

| Section thickness loss % | AR=1.23 uniform corrosion | |
|--------------------------|---------------------------|---|---|---|---|
|                          | Buckling stress N/mm²     | Theoretical crit. Stress | FE crit. Stress | FE Ultimate | b/t | I x 10⁴ mm⁴ |
| 0                        | 244.38                    | 259.3 | 154 | 78 | 16.92 |
| 10                       | 198.05                    | 210.27 | 147.5 | 87 | 12.33 |
| 30                       | 119.81                    | 127.43 | 120.55 | 112 | 5.8 |
| 50                       | 61.1                      | 65.12 | 87.7 | 156 | 2.11 |
| 70                       | 22                        | 23.48 | 55.48 | 260 | 0.46 |
| 90                       | 2.45                      | 2.61 | 20.05 | 780 | 0.017 |
Table 5.3 Critical & ultimate buckling stresses for uniform corrosion at 10% initial imperfection shear load case

| Section thickness loss % | AR=2 | | | | |
|---|---|---|---|---|
| | Theoretical crit. Stress | FE crit. Stress | FE Ultimate | b/t | I x 10^4 mm^4 |
| 0 | 404.67 | 418.73 | 158.7 | 54 | 11.71 |
| 10 | 327.79 | 339.52 | 157.84 | 60 | 8.54 |
| 30 | 198.29 | 205.78 | 146.8 | 77 | 4.02 |
| 50 | 101.17 | 105.16 | 108.43 | 108 | 1.46 |
| 70 | 36.42 | 37.91 | 65.83 | 180 | 0.32 |
| 90 | 4.05 | 4.22 | 18.03 | 540 | 0.012 |

From the load-displacement point of view, Figures 5.2 & 5.3, the uniform corrosion scenarios showed sufficient stiffness up to 50% thickness loss considering the AR 1.23 plate and up to 30% for the AR 2 plate, beyond that a drop in stiffness begins to be noticed more significantly. Stiffness degradation began to be significant at 70% for both plates with ARs 1.23 and 2. From the 70% thickness loss patterns, the worst case scenarios are observed with strength lost between 75% -99% in terms of critical strength and 33% - 88% considering ultimate strength for both ARs.
Figure 5.2 Load-displacement response for uniform corrosion under shear load AR 1.23

Figure 5.3 Load-displacement response for uniform corrosion under shear load AR 2
From the buckling mode shapes, stress and displacement contours point of view, Figures 5.4 to 5.6 it is observed that they plates developed diagonal tension field band around the central portion of the plates consistent with known concept of diagonal tension band for plates in shear (Rockey and Skaloud, 1972). The plate mode shapes come in pairs, particularly for the uniform corrosion scenarios, one mode being positive and the other negative. At higher modes the plates show greater strength as expected indicating membrane stress mobilisation to withstand the applied stress. The mode shape situation for the AR 2 plate under uniform corrosion at mode one, which is normally taking as representative of the plate critical strength, displayed a single tension field band with associated critical strength of 418.73N/mm² indicating a 3% difference with theory. However, the third and fourth mode shapes displayed double diagonal tension band at 434N/mm² showing 7% difference, Figure 5.7. The third and fourth buckling modes in this instance correspond to the global plate type behaviour for this AR 2 and the associated critical stress of 434N/mm² is within acceptable limit of percentage difference.

Figure 5.4 Linear elastic buckling mode shape under shear load for As-new model AR = 1.23
Figure 5.5 Stress deformation contour under shear load for As-new model AR = 1.23 at ultimate stress stage

Figure 5.6 Displacement contour under shear load for As-new model AR = 1.23 at ultimate stress stage
Figure 5.7 Linear elastic buckling mode shapes (modes 1 & 3) under shear load for As-new model AR = 2

Figure 5.8 Stress deformation contour under shear load for As-new model AR = 2 at ultimate stress stage
Figure 5.9 Displacement contour under shear load for As-new model AR = 2 at ultimate stress stage

Figure 5.10 compares the critical and ultimate shear strength reduction factor as a function of percentage section thickness loss for the two ARs of 1.23 & 2 for the uniform corrosion scenario. It is noted that although the plates have different ARs, their elastic behaviours seem quite similar as both plots merge together perfectly. However, they show different inelastic behaviour only merging at around 85% thickness loss. It is evident that both the plate critical and ultimate strength decreases as the level of corrosion deterioration increases. Although, for the AR 2 plate, it shows an initial gradual decrease as indicated by the plateau within the first 30% thickness loss.
Figure 5.10 Critical & ultimate shear strength reduction factor comparison as a function of percentage section thickness loss for uniform corrosion AR 1.23 & 2

5.3.1 Non-uniform corrosion scenario
As with the compressive load cases a total of another 120 models were simulated for the shear response under these corrosion configurations 1 & 2. The same corrosion patterns used for the compressive load models were adopted for the shear load models. The boundary conditions as defined in section 5.2.1 and Tables 5.1 are however different from those used for the compressive load situation.

5.3.2 Configuration 1 bottom corrosion
Tables 5.4 and 5.5 present results of non-uniform corrosion simulations at 10% initial imperfection under shear load for the sixty models under configuration 1. Table 5.4 is for a plate with aspect ratio 1.23 while Table 5.5 for aspect ratio of 2.
Table 5.4 Critical & ultimate buckling stress for non-uniform corrosion at 10% initial imperfection shear load case configuration 1

<table>
<thead>
<tr>
<th>Thickness loss %</th>
<th>AR=1.23 non-uniform corrosion configuration 1</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Buckling stress N/mm²</td>
<td></td>
</tr>
<tr>
<td></td>
<td>condition 1 (0.1b)</td>
<td>condition 2 (0.3b)</td>
</tr>
<tr>
<td></td>
<td>FE critical</td>
<td>FE ultimate</td>
</tr>
<tr>
<td>10</td>
<td>243.23</td>
<td>152.2</td>
</tr>
<tr>
<td>30</td>
<td>153.31</td>
<td>130.77</td>
</tr>
<tr>
<td>50</td>
<td>83.87</td>
<td>98.44</td>
</tr>
<tr>
<td>70</td>
<td>35.04</td>
<td>62.43</td>
</tr>
<tr>
<td>90</td>
<td>6.83</td>
<td>19.21</td>
</tr>
</tbody>
</table>

Table 5.5 Critical & ultimate buckling stress for non-uniform corrosion at 10% initial imperfection shear load case configuration 1

<table>
<thead>
<tr>
<th>Thickness loss %</th>
<th>AR=2 non-uniform corrosion configuration 1</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Buckling stress N/mm²</td>
<td></td>
</tr>
<tr>
<td></td>
<td>condition 1 (0.1b)</td>
<td>condition 2 (0.3b)</td>
</tr>
<tr>
<td></td>
<td>FE critical</td>
<td>FE ultimate</td>
</tr>
<tr>
<td>10</td>
<td>385.78</td>
<td>157</td>
</tr>
<tr>
<td>30</td>
<td>311.22</td>
<td>138.71</td>
</tr>
<tr>
<td>50</td>
<td>223.67</td>
<td>104.73</td>
</tr>
<tr>
<td>70</td>
<td>127.86</td>
<td>66.25</td>
</tr>
<tr>
<td>90</td>
<td>41.83</td>
<td>37.31</td>
</tr>
</tbody>
</table>

Similar to the earlier observation for the uniform corrosion scenario that the linear elastic results for plates with higher aspect ratio seems higher than those of lower aspect ratio, the same situation appears to be repeated here. However, there seem to be only a marginal difference when considering the nonlinear ultimate results, as shown in Tables 5.4 & 5.5. This situation repeats throughout all the different thicknesses and depth combinations. Again, comparing the buckling strength of individual thickness-depth reduction percentage category with that of other thickness-depth reduction percentage categories, it is observed that strength degrades with higher percentage thickness-depth combinations. However, a look at Table 5.5 for example will show that for the AR 2 plate, the 30%-50% higher corrosion pattern for that group seem to follow a strength enhancement pattern given the strength improvement of
152.02N/mm² compared to the lower strength of 138.71N/mm² and 136.56N/mm² corresponding to lower corrosion patterns of 30%-10% and 30%-30% thickness-depth losses of the same group, respectively. It is logically expected to see a decrease in strength but an increase is observed when compared to the behaviour shown by the AR 1.23 plate results in Table 5.4. The same is observed for the 50%-30% when compared to the 50%-10%.

The two situations described above, where there is improvement in strength at higher aspect ratio and at higher thickness-depth reduction at some thickness-depth combination categories may be due to geometric nonlinear effects that disadvantage one pattern against another. The geometric effects in these cases seem to be a plus on geometric stiffness contribution to the overall stiffness. On the whole generally, as expected, strength is seen to be lost with increased corrosion degradation.

The load-displacement behaviour for this corrosion pattern configuration 1 condition 1 AR 1.23 showed sufficient stiffness for the following corrosion patterns: 10%-10% and 30%-10% see Figure 5.11. Stiffness began to degrade significantly at 50%-10% up to 90%-10%, similar to the uniform corrosion case that stiffness started degrading at 50% thickness loss (Figure 5.2). For the AR 2 configuration 1 condition 1, stiffness seems not to degrade except for the 90%-10% scenario see Figure 5.12. The other noticeable difference for the AR 2 configuration 1condition 1 plate appears to be the plateau achieved by the 10%-10% and 90%-10% patterns. All other corrosion patterns under this configuration behaved similarly showing a drop in ultimate strength with each increase in corrosion degree. Similar characteristics is observed with the remaining configuration conditions i.e. configuration 1 conditions 2 & 3 for both plates AR 1.23 and 2, see Figures 5.13 to 5.16.
Figure 5.11 Load-displacement response for non-uniform corrosion (bottom) under shear load AR 1.23 configuration 1 condition 1

Figure 5.12 Load-displacement response for non-uniform corrosion (bottom) under shear load AR 2 configuration 1 condition 1
Figure 5.13 Load-displacement response for non-uniform corrosion (bottom) under shear load AR 1.23 configuration 1 condition 2

Figure 5.14 Load-displacement response for non-uniform corrosion (bottom) under shear load AR 2 configuration 1 condition 2
Figure 5.15 Load-displacement response for non-uniform corrosion (bottom) under shear load AR 1.23 configuration 1 condition 3

Figure 5.16 Load-displacement response for non-uniform corrosion (bottom) under shear load AR 2 configuration 1 condition 3
Figures 5.17 to 5.22 plotted based on input from Tables B.1b to B.1g in appendix (B) show the critical and ultimate shear strength reduction factors as a function of corrosion volume loss ratio (cor/uncor) for both ARs (1.23 & 2) under non-uniform corrosion scenario configuration 1. It can be seen from the Figures that strength indeed drops with increase corrosion deterioration. For the configuration 1 conditions (1, 2 & 3) AR =1.23, it is observed that, they seem to display similar behavioural characteristics which is not so much the case with the AR =2 plate conditions for ultimate strength where conditions 2 & 3 show the formation of a king and dentation at 0.15 and 0.25 volume ratio losses . These volume ratio losses correspond to thickness-depth losses of 50%-30% & 30%-50% respectively.

![Diagram showing critical & ultimate shear strength reduction factors](image)

**Figure 5.17 Critical & ultimate shear strength reduction factors as a function of corrosion volume loss ratio configuration 1 (0.1b) AR 1.23**
Figure 5.18 Critical & ultimate shear strength reduction factors as a function of corrosion volume loss ratio configuration 1 (0.3b) AR 1.23

Figure 5.19 Critical & ultimate shear strength reduction factors as a function of corrosion volume loss ratio configuration 1 (0.5b) AR 1.23
Figure 5.20 Critical & ultimate shear strength reduction factor as a function of corrosion volume loss configuration 1 (0.1b) AR 2

Figure 5.21 Critical & ultimate shear strength reduction factor as a function of corrosion volume loss configuration 1 (0.3b) AR 2
Figures 5.23 to 5.31 show some of the buckling modes, deformed stress and displacement contour shapes of configuration 1 conditions 1, 2 & 3 corrosion patterns AR =1.23 under shear load. The mode shapes indicated some modifications they plates undergo under varying degree of corrosion. The expected linear mode shapes seem to undergo gradual changes or transition except at higher deterioration levels. For example, at 50%-30% thickness-depth loss Figure 5.26 it is observed that at the bottom right hand corner of the plate a sign of the development of a new half-wave tension band is noticed which progressed at 90%-50% thickness-depth loss with clear modification noticed Figure 5.29. Again, they shapes indicate that changes are more readily seen on the stress contours with increasing levels of corrosion deterioration as well the displacement contours. This is evident on Figures 5.24, 5.27 & 5.30 which shows progression of Von-mises stress on the plate stress contours. The same applies to the displacements on the displacement contours on Figures 5.25, 5.28 & 5.31.
Figure 5.23 Linear elastic buckling mode shape under shear load for configuration 1 (10% - 10%) model AR = 1.23

Figure 5.24 Stress deformation contour under shear load for configuration 1 (10% - 10%) model AR = 1.23 at ultimate stress stage
Figure 5.25 Displacement contour under shear load for configuration 1 (10%-10%) model AR = 1.23 at ultimate stress stage

Figure 5.26 Linear elastic buckling mode shape under shear load for configuration 1 (50%-30%) model AR = 1.23
Figure 5.27 Stress deformation contour under shear load for configuration 1 (50%-30%) model AR = 1.23 at ultimate stress stage

Figure 5.28 Displacement contour under shear load for configuration 1 (50%-30%) model AR = 1.23 at ultimate stress stage
Figure 5.29 Linear elastic buckling mode shape under shear load for configuration 1 (90% - 50%) model AR = 1.23

Figure 5.30 Stress deformation contour under shear load for configuration 1 (90% - 50%) model AR = 1.23 at ultimate stress stage
However, the situation appears somewhat different considering the AR 2 plates. These plates are seen to show noticeable modifications from the linear elastic buckling modes. For example, at configuration 1 (30%-10%) corrosion pattern Figure 5.32, they linear mode shape is seen to modify from what was the first mode at the as-new scenario, Figure 5.7. This mode change continues with higher deterioration levels as can be seen on associated Figures 5.35 to 5.52. For the Von-mises and displacements, a similar behavioural characteristics with AR 1.23 plates is seen where stress levels and displacements progresses with increased level of corrosion.

**Figure 5.31** Displacement contour under shear load for configuration 1 (90%-50%) model AR = 1.23 at ultimate stress stage

**Figure 5.32** Linear elastic buckling mode shape under shear load for configuration 1 (30%-10%) model AR = 2 (modes 1 & 3)
Figure 5.33 Stress deformation contour under shear load for configuration 1 (30%-10%) model AR = 2 at ultimate stress stage

Figure 5.34 Displacement contour under shear load for configuration 1 (30%-10%) model AR=2 at ultimate stress stage
Figure 5.35 Linear elastic buckling mode shape under shear load for configuration 1 (50%-10%) model AR = 2

Figure 5.36 Stress deformation contour under shear load for configuration 1 (50%-10%) model AR = 2 at ultimate stress stage
Figure 5.37 Displacement contour under shear load for configuration 1 (50%-10%) model AR=2 at ultimate stress stage

Figure 5.38 Linear elastic buckling mode shape under shear load for configuration 1 (70%-10%) model AR = 2
Figure 5.39 Stress deformation contour under shear load for configuration 1 (70%–10%) model AR = 2 at ultimate stress stage

Figure 5.40 Displacement contour under shear load for configuration 1 (70%–10%) model AR=2 at ultimate stress stage
Figure 5.41 Linear elastic buckling mode shape under shear load for configuration 1 (90%-10%) model AR = 2

Figure 5.42 Stress deformation contour under shear load for configuration 1 (90%-10%) model AR = 2 at ultimate stress stage
Figure 5.43 Displacement contour under shear load for configuration 1 (90% - 10%) model AR=2 at ultimate stress stage

Figure 5.44 Linear elastic buckling mode shape under shear load for configuration 1 (70% - 30%) model AR = 2
Figure 5.45 Stress deformation contour under shear load for configuration 1 (70%-30%) model AR = 2 at ultimate stress stage

Figure 5.46 Displacement contour under shear load for configuration 1 (70%-30%) model AR=2 at ultimate stress stage
Figure 5.47 Linear elastic buckling mode shape under shear load for configuration 1 (90% - 30%) model AR = 2

Figure 5.48 Stress deformation contour under shear load for configuration 1 (90% - 30%) model AR = 2 at ultimate stress stage
Figure 5.49 Displacement contour under shear load for configuration 1 (90%-30%) model AR=2 at ultimate stress stage

Figure 5.50 Linear elastic buckling mode shape under shear load for configuration 1 (70%-50%) model AR = 2
Figure 5.51 Stress deformation contour under shear load for configuration 1 (70%-50%) model AR = 2 at ultimate stress stage

Figure 5.52 Displacement contour under shear load for configuration 1 (70%-50%) model AR=2 at ultimate stress stage

5.3.3 Configuration 2 left, right edges and bottom corrosion
Results from Tables 5.6 and 5.7 show a general decrease in both critical and ultimate strength with thickness-depth loss. However, a few of the corrosion patterns as defined by the relevant percentages indicated the reversed. For example, in Table 5.6 for the AR 1.23 plate the 30%-10% and 10%-10% patterns under critical strength show some improvement in strength with corrosion loss. In terms of ultimate strength, improvement in strength is seen for the 30%-10% and 30%-30% patterns. Also, Table 5.7 for the AR 2 plate show that between 10%-30% and 10%-50% patterns there is strength increase in ultimate strength as well as between 30%-%
30% and 30%-50%, the 50% group and between 70%-10% and 70%-30%. What seem not to be clear though, is that, this strength improvement for the AR 1.23 plate does not follow a particular order or pattern to warrant a conclusion as to why the strength improvement in critical strength. However, the AR 2 plate scenario to an extent seem consistent in a number of patterns and on ultimate strength. A careful observation appears to reveal that increase slenderness may be the reason for the strength enhancement. Again, this improvements in strength could possibly be due to geometrical nonlinearities of the plates as they deteriorate. It appears, some of the corroded geometrical configurations or shapes have some beneficial effect on stiffness and strength.

**Table 5.6** Critical & ultimate buckling stress for non-uniform corrosion at 10% initial imperfection shear load case (left, right & bottom corrosion)

<table>
<thead>
<tr>
<th>Thickness loss %</th>
<th>AR=1.23 non-uniform corrosion configuration 2</th>
<th></th>
<th></th>
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<tbody>
<tr>
<td></td>
<td>Buckling stress N/mm²</td>
<td>condition 1 (0.1b)</td>
<td>condition 2 (0.3b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FE critical</td>
<td>FE ultimate</td>
</tr>
<tr>
<td>10</td>
<td>265.51</td>
<td>160.47</td>
<td>252.21</td>
</tr>
<tr>
<td>30</td>
<td>266.91</td>
<td>146.73</td>
<td>230.85</td>
</tr>
<tr>
<td>50</td>
<td>229.6</td>
<td>92.49</td>
<td>197.35</td>
</tr>
<tr>
<td>70</td>
<td>88.97</td>
<td>37.47</td>
<td>88.63</td>
</tr>
<tr>
<td>90</td>
<td>17.35</td>
<td>13.19</td>
<td>5.68</td>
</tr>
</tbody>
</table>

**Table 5.7** Critical & ultimate buckling stress for non-uniform corrosion at 10% initial imperfection shear load case (left, right & bottom corrosion)

<table>
<thead>
<tr>
<th>Thickness loss %</th>
<th>AR=2 non-uniform corrosion configuration 2</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Buckling stress N/mm²</td>
<td>condition 1 (0.1b)</td>
<td>condition 2 (0.3b)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>FE critical</td>
<td>FE ultimate</td>
</tr>
<tr>
<td>10</td>
<td>430.04</td>
<td>172.89</td>
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</tr>
<tr>
<td>30</td>
<td>387.14</td>
<td>128.2</td>
<td>344.15</td>
</tr>
<tr>
<td>50</td>
<td>235.89</td>
<td>62.46</td>
<td>229.92</td>
</tr>
<tr>
<td>70</td>
<td>65.26</td>
<td>24.72</td>
<td>65.07</td>
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<tr>
<td>90</td>
<td>3.12</td>
<td>8.1</td>
<td>3.02</td>
</tr>
</tbody>
</table>

Under configuration 2 AR 1.23, the load-displacement response for condition 1 for example, seem to display variable stiffness distribution for a number of the corrosion patterns involved.
Figure 5.53. They patterns involved appear to show that stiffness degrades with increased corrosion deterioration. This situation seem a bit different with the other conditions (1 & 2) for the same configuration 2 where steady stiffness appears to be maintained Figures 5.54 & 5.55.

**Figure 5.53 Load-displacement response for non-uniform corrosion (left, right & bottom) under shear load AR 1.23 configuration 2 condition 1**
Figure 5.54 Load-displacement response for non-uniform corrosion (left, right & bottom) under shear load AR 1.23 configuration 2 condition 2

Figure 5.55 Load-displacement response for non-uniform corrosion (left, right & bottom) under shear load AR 1.23 configuration 2 condition 3
Considering the load-displacement relationship for the AR 2 plates configuration 2 conditions 1, 2 & 3 Figures 5.56 to 5.58, a much steady stiffness is observed except at higher deterioration levels.

Figure 5.56 Load-displacement response for non-uniform corrosion (left, right & bottom) under shear load AR 2 configuration 2 condition 1
Figure 5.57 Load-displacement response for non-uniform corrosion (left, right & bottom) under shear load AR 2 configuration 2 condition 2

Figure 5.58 Load-displacement response for non-uniform corrosion (left, right & bottom) under shear load AR 2 configuration 2 condition 3
Figures 5.59 to 5.64 plotted based on Tables B.2a to B.2f in appendix (B) show the critical and ultimate shear buckling strength reduction factors as a function of corrosion volume loss ratio for non-uniform corrosion configuration 2 AR 1.23 and 2. As expected, strength generally decreased with corrosion loss but the situation vary for the different corrosion conditions. For example, conditions 1 & 2 Figures 5.59 & 5.60 show that significant drop is noticed beyond the 10% volume loss where for example ultimate strength drops to about 60%. Condition 3 show a continuous steady drop from the onset of both critical and ultimate strength Figure 5.61.

![Figure 5.59 Critical and ultimate shear strength reduction factor as a function of corrosion volume loss for non-uniform corrosion configuration 2 condition 1 AR 1.23](image-url)
Figure 5.60 Critical and ultimate shear strength reduction factor as a function of corrosion volume loss for non-uniform corrosion configuration 2 condition 2 AR 1.23

Figure 5.61 Critical and ultimate shear strength reduction factor as a function of corrosion volume loss for non-uniform corrosion configuration 2 condition 3 AR 1.23
Looking at the AR 2 plate behaviour in terms of reduction factors, it is observed that beyond the 10% volume loss for configuration 2 condition 1 both critical and ultimate strength reduce to about 56% and 39% respectively. While condition 2 and 3 show significant reduction beyond 20% with 55% and 67% of critical strength remaining and ultimate strength indicating 39% and 53% remaining strength respectively Figures 5.62 to 5.64.

**Figure 5.62** Critical and ultimate shear strength reduction factor as a function of corrosion volume loss for non-uniform corrosion configuration 2 condition 1 AR 2
Figure 5.63 Critical and ultimate shear strength reduction factor as a function of corrosion volume loss for non-uniform corrosion configuration 2 condition 2 AR 2

Figure 5.64 Critical and ultimate shear strength reduction factor as a function of corrosion volume loss for non-uniform corrosion configuration 2 condition 3 AR 2
From the buckling modes, stress and displacement contours, it is observed that buckling mode shapes become modified with contours flowing towards the top right hand corner of the plate. For example, considering a number of corrosion patterns configuration 2 condition 1 AR 1.23 scenarios Figures 5.65 to 5.79, it can be seen that the diagonal tension band kept changing until it localised one side of the plate when looking from the linear buckling mode shapes. Of course, the nonlinear deformed shapes in terms of stress and displacement contours did change too with the corroded edges bearing high stress distribution and possibly displacements too. The above characteristic behaviour is observed with further deterioration patterns as demonstrated by the associated Figures representing thickness-depth patterns 50%-10%, 70%-10%, 70%-30%, 90%-30% and 70%-50%.

Figure 5.65 Linear elastic buckling mode shape under shear load for configuration 2 (50%-10%) model AR = 1.23
Figure 5.66 Stress deformation contour under shear load for configuration 2 (50%-10%) model
AR = 1.23 at ultimate stress stage

Figure 5.67 Displacement contour under shear load for configuration 2 (50%-10%) model
AR=1.23 at ultimate stress stage
Figure 5.68 Linear elastic buckling mode shape under shear load for configuration 2 (70% - 10%) model AR = 1.23

Figure 5.69 Stress deformation contour under shear load for configuration 2 (70% - 10%) model AR = 1.23 at ultimate stress stage
Figure 5.70 Displacement contour under shear load for configuration 2 (70%-10%) model AR=1.23 at ultimate stress stage

Figure 5.71 Linear elastic buckling mode shape under shear load for configuration 2 (70%-30%) model AR = 1.23
Figure 5.72 Stress deformation contour under shear load for configuration 2 (70%-30%) model
AR = 1.23 at ultimate stress stage

Figure 5.73 Displacement contour under shear load for configuration 2 (70%-30%) model
AR=1.23 at ultimate stress stage
Figure 5.74 Linear elastic buckling mode shape under shear load for configuration 2 (90\%-30\%) model AR = 1.23

Figure 5.75 Stress deformation contour under shear load for configuration 2 (90\%-30\%) model AR = 1.23 at ultimate stress stage
Figure 5.76 Displacement contour under shear load for configuration 2 (90%-30%) model AR=1.23 at ultimate stress stage

Figure 5.77 Linear elastic buckling mode shape under shear load for configuration 2 (70%-50%) model AR = 1.23
Figure 5.78 Stress deformation contour under shear load for configuration 2 (70% - 50%) model
AR = 1.23 at ultimate stress stage

Figure 5.79 Displacement contour under shear load for configuration 2 (70% - 50%) model
AR=1.23 at ultimate stress stage

A similar behavioural characteristics is observed for the AR 2 plates as can be seen in Figures 5.80 to 5.85. It is also observed from the associated legends of both the AR 1.23 and 2 plates that the Von-mises stress at ultimate stress stage is mostly above the material yield stress of 275N/mm².
Figure 5.80 Linear elastic buckling mode shape under shear load for configuration 2 (10%-10%) model AR = 2

Figure 5.81 Stress deformation contour under shear load for configuration 2 (10%-10%) model AR = 2 at ultimate stress stage
Figure 5.82 Displacement contour under shear load for configuration 2 (10%-10%) model AR=2 at ultimate stress stage

Figure 5.83 Linear elastic buckling mode shape under shear load for configuration 2 (50%-10%) model AR = 2
Figure 5.84 Stress deformation contour under shear load for configuration 2 (50%-10%) model AR = 2 at ultimate stress stage

Figure 5.85 Displacement contour under shear load for configuration 2 (50%-10%) model AR=2 at ultimate stress stage

5.4 Summary
This chapter presented results of FE simulations of web plates under shear loading. Certain deformation characteristics and stress distribution were observed and described. Generally, it is observed that the critical and ultimate strength of the plates reduces with corrosion degradation. This strength or capacity loss can be associated with the type of buckling mode, stress and displacement contour modifications observed on the various models as the plates suffer increasing deterioration. The geometrical imperfection introduced into the plate models by the corrosion phenomena is responsible for the kind of stress distribution...
concentration leading to either strength reduction or enhancement in some cases. It is worth noting that both the critical and ultimate strength reduction factors for the uniform corrosion scenarios looks a bit smoother than the ones for the non-uniform corrosion scenarios where in some cases a king, dentation or overlaps are formed, see Figures 5.21, 5.22 & 5.60. It may be explained that the non-uniform scenarios are not smooth because of the presence of geometric nonlinearities due to corrosion degradation which is mostly random in distribution over the plate not necessarily because of initial plate imperfection. The next chapter will involve quantification of corrosion deterioration using available dose response functions to determine the potential damage over time based on projected climate emission scenarios. The potential damages will be related to this strength performance mapping in a novel way so as to capture the potential effect of climate change over time on the plate element.
CHAPTER SIX

Climate change effects on buckling strength of plates

6.1 Introduction
The aim of this chapter is to quantify corrosion damage as a result of changes in environmental and atmospheric pollution through the relevant dose-response functions (DRF) provided in the ISO standards, the Klinesmith et al. (2007) and the Kallias et al. (2016) models thereby linking climate change with buckling strength. This corrosion loss will be estimated based on projected changes in the relevant climate change parameters consistent with the IPCC and UKCP09 defined emission scenarios. The estimated projected corrosion loss over time, considering the lifespan of the structural asset, will be related to the buckling strength degradation derived from the full corrosion performance mapping carried out in chapters four and five through the finite element method.

6.2 Climate change background
The Intergovernmental Panel on Climate Change (IPCC) is the international body that seeks to collate relevant scientific information based on peer-reviewed literature so as to enhance understanding of the physical processes in the climate system. This body comes up with Assessment Reports focusing on key findings and also highlighting what is new since the last report. Since the last IPCC’s Third Assessment Report (TAR) in 2001, listed as (Foster, 2001), significant progress has been made in understanding past and recent climate change phenomena as well as future projections relative to the changes. This progress has led to the release of the Fourth Assessment Report (FAR) in 2007 listed as (Change, 2007). This is made possible because of the availability of large amount of new data, more sophisticated analyses of data, improvements in the understanding and simulation of physical processes in
climate models and a more extensive exploration of uncertainty ranges in models outcomes (Solomon, 2007). With this development improved confidence in climate science has been achieved.

Figure 6.1 shows the temperature evolution over time and future predictions under three different scenarios, which are representations of optimistic and pessimistic scenarios with respect to how emissions will evolve over time. These scenarios will be discussed in detail later on.

![Figure 6.1 Increase in global temperature for the different emission scenarios using a pre-industrial baseline of 1760 (Source: UK climate projections, 2009)](image)

6.2.1 Emission scenarios

The extent of climate change is believed to depend to a large degree on how successful the world cuts down greenhouse gas emissions (UKCP, 2009). An understanding of how the world climate might change is important in helping to prepare against the impact. A good evidence of this change including a measure of the uncertainties involved will help in the consideration of the risk that this changing climate might pose. Therefore, helping in planning increase resilience and risk reduction (UKCP, 2009). It is in the light of this, that the UK climate projection 2009 have been developed. Again, the United Nations Framework Convention on Climate Change (UNFCCC) negotiation at Copenhagen sort to reach an
international agreement to limit global greenhouse gas emission but, the policy was limited in that it lack what it takes to enforce the global agreement. Hence, the Conference of Parties (COP 21) held in Paris 2015 to enable enforcement. The Paris conference was tailored towards an agreement that will limit global temperature increases to no more than 2°C above the pre-industrial levels, beyond which it is believed the risk of climate change may assume a greater risk level. So the need to take action to limit emission to further avoid dangerous climate change. The UK Committee on climate change feels, global emissions need to peak by 2016 and then fall by 2050 to at least 50% below the 1990 levels if the 2°C limiting temperature is to be achieved.

UKCP09 categorises the greenhouse gas emission scenarios into three different pathways showing how a range of factors, such as population, economic growth and energy usage, might change over time. Figure 6.2 diagrammatically explain the emission story lines. The three emission scenarios are also shown on Figure 6.3 below namely, high, medium and low scenarios. These scenarios are based on expected projections provided in the Special Report on Emissions (SRES) developed by the IPCC in 2000.
Figure 6.2 Emissions story line into families (Source: UK climate projection report, 2009)
6.2.1.1 High emission scenario
Following the high emission pathway where heavy reliance is on the use of fossil fuel, a global temperature rise of over 5.5°C by 2100 is projected as compared with the pre-industrial period. The implication of this for the UK is that in the South West of England for example, by 2080’s an average summer temperature rise of 5°C is expected.

6.2.1.2 Medium emission scenario
This scenario pathway describes a future world of very rapid economic growth and a growing population peaking in 2050 at say nearly 9 billion. The pathway also supposes a continuation of the usage of fossil fuels but, with a substitution to renewable energy sources at some point in time. It also assumes rapid introduction of new and efficient technologies driven by market forces. The implication of this to the UK particularly the South East of England is a summer temperature of 3.9°C by 2080’s.

6.2.1.3 Low emission scenario
This pathway assumes a strong shift in human activities away from fossil fuel. The low emission path implies a rise of about 3°C in Southern England by 2080’s.
6.2.2 UK climate change projections
Tables 6.2 to 6.6 show published projected temperature, precipitation and relative humidity data relative to the 1961-1990 baseline for the 2050’s and 2080’s for the UK. These data are a helpful resource in estimating the expected corrosion degradation during the life span of an infrastructural asset, through the use of the dose-response functions.

Table 6.1 Highest and lowest changes in mean daily temperature, mean daily maximum temperature and mean daily minimum temperature (°C) in winter and summer, by the 2080’s, relative to 1961-1990 (UK Climate projection, 2009)

<table>
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<th>Mean temperature, summer</th>
<th>Mean daily maximum temperature, winter</th>
<th>Mean daily maximum temperature, summer</th>
<th>Mean daily minimum temperature, winter</th>
<th>Mean daily minimum temperature, summer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10% 50% 90%</td>
<td>10% 50% 90%</td>
<td>10% 50% 90%</td>
<td>10% 50% 90%</td>
<td>10% 50% 90%</td>
<td>10% 50% 90%</td>
</tr>
<tr>
<td>High emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>2.2 3.8 5.8</td>
<td>2.9 5.3 8.4</td>
<td>1.6 2.4 3.2</td>
<td>0.8 2.5 4.4</td>
<td>1.7 2.9 4.1</td>
<td>0.8 2.5 4.4</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>1.0 2.1 3.5</td>
<td>1.6 3.1 5.0</td>
<td>1.1 2.3 3.5</td>
<td>0.8 2.5 4.4</td>
<td>1.7 2.9 4.1</td>
<td>0.8 2.5 4.4</td>
</tr>
<tr>
<td>Medium emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>1.7 3.1 4.8</td>
<td>2.2 4.2 6.8</td>
<td>1.3 2.9 4.1</td>
<td>0.6 2.1 3.7</td>
<td>1.4 2.9 4.8</td>
<td>0.6 2.1 3.7</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>0.8 1.8 3.1</td>
<td>1.2 2.5 4.1</td>
<td>0.8 2.0 3.4</td>
<td>0.6 2.1 3.7</td>
<td>1.4 2.9 4.8</td>
<td>0.6 2.1 3.7</td>
</tr>
<tr>
<td>Low emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>1.5 2.7 4.1</td>
<td>1.4 3.1 5.3</td>
<td>1.3 2.6 3.2</td>
<td>0.7 2.1 3.9</td>
<td>1.4 2.9 4.8</td>
<td>0.7 2.0 3.3</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>0.8 1.7 2.7</td>
<td>0.8 1.9 3.2</td>
<td>0.9 1.8 3.0</td>
<td>0.7 2.1 3.9</td>
<td>1.4 2.9 4.8</td>
<td>0.7 2.0 3.3</td>
</tr>
</tbody>
</table>
Table 6.2 Changes in daily mean (summer & winter averages), and summer-mean daily maximum & minimum temperatures, averaged over administrative regions by 2050s under medium emissions scenario (UK Climate projection, 2009)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean temperature, winter °C</th>
<th>Mean temperature, summer °C</th>
<th>Mean daily maximum temperature, summer °C</th>
<th>Mean daily minimum temperature, summer °C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability level</td>
<td>10%</td>
<td>50%</td>
<td>90%</td>
<td>Wider range</td>
</tr>
<tr>
<td>North Scotland</td>
<td>0.6</td>
<td>1.7</td>
<td>2.8</td>
<td>0.6</td>
</tr>
<tr>
<td>East Scotland</td>
<td>0.7</td>
<td>1.7</td>
<td>2.9</td>
<td>0.6</td>
</tr>
<tr>
<td>West Scotland</td>
<td>1.0</td>
<td>1.9</td>
<td>3.0</td>
<td>0.8</td>
</tr>
<tr>
<td>N Ireland</td>
<td>0.9</td>
<td>1.7</td>
<td>2.7</td>
<td>0.6</td>
</tr>
<tr>
<td>Isle of Man</td>
<td>0.9</td>
<td>1.8</td>
<td>2.7</td>
<td>0.7</td>
</tr>
<tr>
<td>NE England</td>
<td>1.0</td>
<td>2.0</td>
<td>3.1</td>
<td>0.8</td>
</tr>
<tr>
<td>NW England</td>
<td>1.0</td>
<td>2.0</td>
<td>3.0</td>
<td>0.8</td>
</tr>
<tr>
<td>Yorkshire &amp; Humber</td>
<td>1.1</td>
<td>2.1</td>
<td>3.3</td>
<td>0.9</td>
</tr>
<tr>
<td>East Midlands</td>
<td>1.1</td>
<td>2.2</td>
<td>3.4</td>
<td>0.9</td>
</tr>
<tr>
<td>West Midlands</td>
<td>1.2</td>
<td>2.1</td>
<td>3.2</td>
<td>0.9</td>
</tr>
<tr>
<td>Wales</td>
<td>1.1</td>
<td>2.0</td>
<td>3.1</td>
<td>0.8</td>
</tr>
<tr>
<td>East England</td>
<td>1.1</td>
<td>2.2</td>
<td>3.4</td>
<td>0.9</td>
</tr>
<tr>
<td>London</td>
<td>1.2</td>
<td>2.2</td>
<td>3.5</td>
<td>0.9</td>
</tr>
<tr>
<td>SE England</td>
<td>1.1</td>
<td>2.2</td>
<td>3.4</td>
<td>0.9</td>
</tr>
<tr>
<td>SW England</td>
<td>1.1</td>
<td>2.1</td>
<td>3.2</td>
<td>0.8</td>
</tr>
<tr>
<td>Channel Isles</td>
<td>1.1</td>
<td>2.0</td>
<td>3.1</td>
<td>0.8</td>
</tr>
</tbody>
</table>
Table 6.3 Changes in annual winter and summer mean precipitation averaged over administrative regions by 2050’s under medium emissions scenario (UK Climate projection report, 2009)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Annual mean precipitation %</th>
<th>Winter mean precipitation %</th>
<th>Summer mean precipitation %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10%</td>
<td>50%</td>
<td>90%</td>
</tr>
<tr>
<td>North Scotland</td>
<td>-6</td>
<td>0</td>
<td>+5</td>
</tr>
<tr>
<td>East Scotland</td>
<td>-4</td>
<td>0</td>
<td>+5</td>
</tr>
<tr>
<td>West Scotland</td>
<td>-6</td>
<td>0</td>
<td>+5</td>
</tr>
<tr>
<td>Northern Ireland</td>
<td>-3</td>
<td>0</td>
<td>+3</td>
</tr>
<tr>
<td>Isle of Man</td>
<td>-5</td>
<td>0</td>
<td>+4</td>
</tr>
<tr>
<td>North East England</td>
<td>-4</td>
<td>0</td>
<td>+5</td>
</tr>
<tr>
<td>North West England</td>
<td>-5</td>
<td>0</td>
<td>+6</td>
</tr>
<tr>
<td>Yorkshire &amp; Humber</td>
<td>-3</td>
<td>0</td>
<td>+4</td>
</tr>
<tr>
<td>East Midlands</td>
<td>-4</td>
<td>0</td>
<td>+6</td>
</tr>
<tr>
<td>West Midland</td>
<td>-4</td>
<td>0</td>
<td>+6</td>
</tr>
<tr>
<td>Wales</td>
<td>-4</td>
<td>0</td>
<td>+5</td>
</tr>
<tr>
<td>East England</td>
<td>-4</td>
<td>0</td>
<td>+5</td>
</tr>
<tr>
<td>London</td>
<td>-4</td>
<td>0</td>
<td>+5</td>
</tr>
<tr>
<td>South East England</td>
<td>-4</td>
<td>0</td>
<td>+6</td>
</tr>
<tr>
<td>South West England</td>
<td>-4</td>
<td>0</td>
<td>+6</td>
</tr>
<tr>
<td>Channel Islands</td>
<td>-4</td>
<td>0</td>
<td>+3</td>
</tr>
</tbody>
</table>
Table 6.4 Highest and lowest changes in annual, winter and summer mean daily precipitation, and in precipitation on wettest day of the season (%) in winter and summer, by the 2080’s, relative to 1961-1990 (UK Climate projection report, 2009)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Mean precipitation, annual</th>
<th>Mean precipitation, winter</th>
<th>Mean precipitation, summer</th>
<th>Precipitation on wettest day of the winter</th>
<th>Precipitation on wettest day of the summer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability level</td>
<td>10%</td>
<td>50%</td>
<td>90%</td>
<td>10%</td>
<td>50%</td>
</tr>
<tr>
<td>High emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>-3</td>
<td>+3</td>
<td>+20</td>
<td>+18</td>
<td>+47</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>-21</td>
<td>+6</td>
<td>+3</td>
<td>-12</td>
<td>-3</td>
</tr>
<tr>
<td>Medium emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>-3</td>
<td>+2</td>
<td>+14</td>
<td>+9</td>
<td>+33</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>-16</td>
<td>-3</td>
<td>-3</td>
<td>-11</td>
<td>-2</td>
</tr>
<tr>
<td>Low emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>-2</td>
<td>+3</td>
<td>+14</td>
<td>+8</td>
<td>+30</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>-12</td>
<td>-1</td>
<td>+3</td>
<td>-11</td>
<td>-2</td>
</tr>
</tbody>
</table>

Table 6.5 Highest and lowest changes in cloud amount (%) and mean relative humidity (%) in winter and summer, relative to 1961-1990 (Source: UK climate projection report, 2009)

<table>
<thead>
<tr>
<th>Variable</th>
<th>Total cloud, winter</th>
<th>Total cloud, summer</th>
<th>Relative humidity, winter</th>
<th>Relative humidity, summer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Probability level</td>
<td>10%</td>
<td>50%</td>
<td>90%</td>
<td>10%</td>
</tr>
<tr>
<td>High emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>-1</td>
<td>+2</td>
<td>+8</td>
<td>+1</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>-10</td>
<td>-4</td>
<td>+1</td>
<td>-39</td>
</tr>
<tr>
<td>Medium emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>-1</td>
<td>+1</td>
<td>+6</td>
<td>0</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>-9</td>
<td>-4</td>
<td>+1</td>
<td>-33</td>
</tr>
<tr>
<td>Low emissions</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Highest change in UK</td>
<td>-1</td>
<td>+1</td>
<td>+5</td>
<td>-1</td>
</tr>
<tr>
<td>Lowest change in UK</td>
<td>-8</td>
<td>-3</td>
<td>+1</td>
<td>-25</td>
</tr>
</tbody>
</table>

6.3 The effects of environmental conditions on atmospheric corrosion
As mentioned earlier, changes in climatic parameters like temperature, relative humidity, time of wetness and atmospheric pollution concentrations such as Sulphur dioxide and Chloride concentration are likely to alter the degradation rates of structural materials. The dose-
response functions offer a pathway towards linking corrosion loss under changing conditions, both environmental as well as pollution concentrations.

6.3.1 Corrosion damage assessment based on BS EN ISO 9223 & 9224: 2012
Chapter two extensively discussed corrosivity of atmospheres; classification, determination and estimation. Data from the International exposure programs ISO CORRAG and MICAT were discussed to show how such data contributed in the deepening of knowledge and adjustment of the existing corrosivity classifications. The works of various researchers (Knotkova and Kreislova, 2007; Tidblad et al., 1990 & 2002; Morcillo, Almeida & Rosales, 1998) were discussed showing how these researchers contributed in the development of new, better and elaborate dose-response functions that are generally valid today for use in a wider regional scale.

BS EN ISO 9223 (2012) classifies atmospheric corrosivity into two categories. Firstly, classification based on corrosivity determination by corrosion rate measurement on standard coupons and secondly base on estimation of corrosivity on the basis of environmental information. This research considers corrosion damage assessment based on environmental information considering expected potential changes in the climate over the service life span of a bridge structure. The dose-response function relative to carbon steel is used as presented in equation (2.13) in chapter two.

Equation (2.13) estimates the first year corrosion while equation (2.36) as adapted from BS EN ISO 9224 (2012) Estimates corrosion attack (D) beyond the first year. Equation (2.36) predicts corrosion loss up to twenty years followed by corrosion product layer protection which slows down the rate of corrosion after the first twenty years. The relationship is exponential within the first twenty years but afterwards it becomes linear within time because the rate of metal loss becomes equal to the rate of loss from the corrosion product layer (BS EN ISO 9224, 2012). In order to capture behaviour after the first twenty years equation (2.36) is differentiated with respect to time to give the rate of corrosion as expressed in equations (2.37) and (2.38). Equation (2.38), expressed below again, is used to estimate the corrosion damage based on long-term exposures.

\[ D(t > 20) = r_{corr}[20^b + b(20^{b-1})(t - 20)] \]  (6.1)
Table 6.7, adopted from the BS EN ISO 9223 (2012), gives values for the term \( b \) \((20^{b-1})\) for the \( b \) values shown in Table 6.8, which is also adopted from the ISO standard. The choice of \( B1 \) and \( B2 \) values is influenced by whether a lower bound, conservative upper bound or upper bound corrosion attack value is required. When a normal damage value is required \( B1 \) is used. However, sometimes it becomes important to estimate a conservative upper bound corrosion attack value after an extended exposure. In that case the \( b \) value required for use in equation (2.36) should be increased to account for uncertainties in the data. One way of doing this is by the addition of two standard deviations to the average \( b \) value so as to obtain a value at the upper 95% confidence level (BS EN ISO 9224, 2012). The values of a few metals standard deviations are given in Table 6.10.

The above procedure is only possible in situations where the first year corrosion attack is known or can be estimated by the expression in ISO 9223 (2012) equation (2.13) in chapter two. The aim is to predict the extent of attack after an extended exposure. The corrosion damage prediction is calculated by substituting the first year corrosion \((r_{corr})\) in equation (2.36). The appropriate \( b \) value can be selected or calculated in accordance with clause 7 in ISO 9224 (2012). In an instance where the long-term metal loss data are known and available, the \( b \) value from the data should be used, otherwise the \( B1 \) value from Table 6.8 for the metal or alloy in question should be selected and used. This \( B1 \) value indicates the relevant \( b \) value to be used in equation (2.36) as seen on Table 6.9.

It should be note that the metal-environment-specific time exponent (\( b \) value) used in equation (2.36) can be categorised into two;

1. \( b \) value based on estimated exposure data and
2. \( b \) value based on \( B1 \) and \( B2 \) values assumed or calculated from the ISO CORRAG program. So, they are termed generalised \( b \) values.

Table 6.9 presents values of the function \( t^b \) for time values up to 100 years with \( B1 \) exponents to simplify the computations. However, the possibility is there for equation (2.36) not to apply to exposures beyond 20 years as described in clause 7 on long-term exposures.

The \( B2 \) values in Table 6.8 are values that define the upper bound of corrosion attack. These values are obtained by the inclusion of the two standard deviations from the ISO CORRAG program.
data. As can be seen, Table 6.9 also provides calculated values for the $t^b$ function up to 100 years using the B2 values for b.

**Table 6.6** Values of $b (20^{b-1})$ adapted from BS EN ISO 9224 (2012)

<table>
<thead>
<tr>
<th>Metal</th>
<th>$b$</th>
<th>$20^b$</th>
<th>$b(20^{b-1})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon steel</td>
<td>B1</td>
<td>0.523</td>
<td>4,791</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>0.575</td>
<td>5,559</td>
</tr>
<tr>
<td>Zinc</td>
<td>B1</td>
<td>0.813</td>
<td>11,422</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>0.873</td>
<td>13,671</td>
</tr>
<tr>
<td>Copper</td>
<td>B1</td>
<td>0.667</td>
<td>7,375</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>0.726</td>
<td>8,803</td>
</tr>
<tr>
<td>Aluminium</td>
<td>B1</td>
<td>0.728</td>
<td>8,854</td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>0.807</td>
<td>11,218</td>
</tr>
</tbody>
</table>

**Table 6.7** Time exponent values for predicting and estimating corrosion attack adapted from BS EN ISO 9224 (2012)

<table>
<thead>
<tr>
<th>Metal</th>
<th>B1</th>
<th>B2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon steel</td>
<td>0.523</td>
<td>0.575</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.813</td>
<td>0.873</td>
</tr>
<tr>
<td>Copper</td>
<td>0.667</td>
<td>0.726</td>
</tr>
<tr>
<td>Aluminium</td>
<td>0.728</td>
<td>0.807</td>
</tr>
</tbody>
</table>
Table 6.8 Metal-environment-specific time exponents for standard metals adapted from BS EN ISO 9224 (2012)

<table>
<thead>
<tr>
<th></th>
<th>Steel</th>
<th>Zinc</th>
<th>Copper</th>
<th>Aluminium</th>
</tr>
</thead>
<tbody>
<tr>
<td>b values</td>
<td>0.523</td>
<td>0.575</td>
<td>0.813</td>
<td>0.873</td>
</tr>
<tr>
<td>r (years)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>2</td>
<td>1.437</td>
<td>1.490</td>
<td>1.757</td>
<td>1.831</td>
</tr>
<tr>
<td>3</td>
<td>1.776</td>
<td>1.881</td>
<td>2.443</td>
<td>2.609</td>
</tr>
<tr>
<td>4</td>
<td>2.065</td>
<td>2.219</td>
<td>3.087</td>
<td>3.354</td>
</tr>
<tr>
<td>5</td>
<td>2.320</td>
<td>2.523</td>
<td>3.701</td>
<td>4.075</td>
</tr>
<tr>
<td>6</td>
<td>2.553</td>
<td>2.802</td>
<td>4.292</td>
<td>4.779</td>
</tr>
<tr>
<td>7</td>
<td>2.767</td>
<td>3.061</td>
<td>4.865</td>
<td>5.467</td>
</tr>
<tr>
<td>8</td>
<td>2.967</td>
<td>3.306</td>
<td>5.423</td>
<td>6.143</td>
</tr>
<tr>
<td>9</td>
<td>3.156</td>
<td>3.537</td>
<td>5.968</td>
<td>6.809</td>
</tr>
<tr>
<td>11</td>
<td>3.505</td>
<td>3.970</td>
<td>7.025</td>
<td>8.112</td>
</tr>
<tr>
<td>12</td>
<td>3.668</td>
<td>4.174</td>
<td>7.540</td>
<td>8.752</td>
</tr>
<tr>
<td>14</td>
<td>3.976</td>
<td>4.561</td>
<td>8.547</td>
<td>10.013</td>
</tr>
<tr>
<td>17</td>
<td>4.401</td>
<td>5.099</td>
<td>10.008</td>
<td>11.863</td>
</tr>
<tr>
<td>20</td>
<td>4.791</td>
<td>5.599</td>
<td>11.422</td>
<td>13.671</td>
</tr>
</tbody>
</table>

- Table continues with similar data for Steel, Zinc, Copper, and Aluminium for different values of Steel.
Table 6.9 Standard deviation of b values adapted from BS EN ISO 9224 (2012)

<table>
<thead>
<tr>
<th>Metal</th>
<th>Standard deviation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon steel</td>
<td>0.0260</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.0300</td>
</tr>
<tr>
<td>Copper</td>
<td>0.0295</td>
</tr>
<tr>
<td>Aluminium</td>
<td>0.0395</td>
</tr>
</tbody>
</table>

6.3.2 Corrosion damage assessment base on the Klinesmith et al. 2007 model

Normally, most of the corrosion models used in predicting corrosion loss do that as a function of time only (Klinesmith et al., 2007). The limitation of these models is that, all the effects and variations related to environmental conditions are being represented by the proportionality constant and the mass loss exponent in the time-dependent model (Klinesmith et al., 2007). Again, the time-dependent models are restricted to evaluating corrosion loss only in the environment in which they models were calibrated, otherwise the model prediction in a different environment will be inaccurate (Klinesmith et al., 2007). Therefore, this limitation led to the need of developing models that can estimate corrosion loss as a function of both environmental conditions and time. Furthermore, this new models are expected to provide the physical rational effects of the environmental parameters on the loss if they must be useful in predicting the design life or the ultimate corrosion loss of the structure.

The environmental parameters of interest here are the time of wetness (TOW), sulphur dioxide concentration, chloride deposition and air temperature, as already mentioned before. The inclusion of these environmental parameters, particularly the TOW, in corrosion loss models will help model prediction of service life of the structure given that the models are no longer restricted to time alone. The challenge is also to establish a relationship that will link TOW and the exposure period over which wetness occurs. It should be noted that most models use TOW to capture the effect of relative humidity in corrosion loss estimation (Klinesmith et al., 2007). In view of the above reasons (Klinesmith et al., 2007) developed a model that integrates time and account for these multi environmental parameters. They model seems to be physically rational, permitting variation of time and possessing flexibility to
allow application in other locations. The integration of both the time function and the environmental adjustment parameters is very useful in that the time function determines the rate of corrosion while the environmental parameters change the rate of corrosion in accordance with the environmental condition. The Klinesmith et al. (2007) model is presented below:

\[ C_{\text{loss}} = A t^B \left( \frac{TOW}{C} \right)^D \left( 1 + \frac{SO_2}{E} \right)^F \left( 1 + \frac{Cl}{G} \right)^H e^{J(T+T_o)} \]  

(6.2)

Where \( C_{\text{loss}} = \) corrosion loss (μm); \( t = \) exposure time (years); \( TOW = \) time of wetness (h/years); \( SO_2 = \) sulphur dioxide concentration (μg/m³); \( Cl = \) chloride deposition rate (mg/m²/day); \( T = \) air temperature (oC); and \( A, B, C, D, E, F, G, H, J \) and \( T_o = \) empirical coefficients (see Table 6.11)

**Table 6.10** Coefficients of equation 6.2 calibrated with the ISO CORRAG Data with \( C = 3800, E = 25, G = 50 \) and \( T_o = 20 \) (Klinesmith et al., 2007)

<table>
<thead>
<tr>
<th>Materials</th>
<th>Types of specimens</th>
<th>Equation coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
</tr>
<tr>
<td>CS</td>
<td>Flat</td>
<td>13.4</td>
</tr>
<tr>
<td>CS</td>
<td>Helix</td>
<td>19.7</td>
</tr>
<tr>
<td>Z</td>
<td>Flat</td>
<td>0.16</td>
</tr>
<tr>
<td>Z</td>
<td>Helix</td>
<td>0.26</td>
</tr>
<tr>
<td>C</td>
<td>Flat</td>
<td>0.46</td>
</tr>
<tr>
<td>C</td>
<td>Helix</td>
<td>0.78</td>
</tr>
<tr>
<td>A</td>
<td>Flat</td>
<td>0.948</td>
</tr>
<tr>
<td>A</td>
<td>Helix</td>
<td>0.27</td>
</tr>
</tbody>
</table>

Note: CS = carbon steel; Z = zinc; C = copper; and A = aluminum.

The \( A t^B \) part of equation (6.2) represents the time function in accordance to (Dean, 1990; Dean and Reiser, 2002; Kucera et al., 1987; Schweitzer, 1999; Townsend, 2002). This part is useful in the computation of the rate of corrosion as a function of exposure time. The rate of corrosion varies with exposure time because of corrosion product layer formation. The remaining part of equation (6.2) represents the series of the environmental adjustments.
parameters: TOW, SO₂, Cl, and T. As mentioned earlier, the role of these parameters is to change the rate of corrosion relative to the environmental conditions.

As mentioned earlier that most models capture the effects of relative humidity (RH) via the TOW, this may not be far from the fact that the TOW is considered the fraction of time the metal surface remains exposed to wetness. In a more simple way, TOW is the time in which the RH is greater than 80% and T greater than 0°C (Dean and Reiser, 2002). In more practical terms, TOW is the number of hours per year in which the RH is greater than 80% and T greater than 0°C.

Tidblad et al. (2000) reported that TOW can vary from nearly zero to 8766 h/year. The coldest regions of the world are observed to have the lowest TOW values. The average annual corrosion rates in these cold regions (Antarctic, Artic, Siberia, Far East, and Alaska) are observed to be very low. However, if the temperature in these cold regions becomes high, TOW will obviously become high as well the corrosion rate. Furthermore, because TOW is a parameter that depends on climatic conditions as well as the nature of the material surface, it should be relatively easy to compute from the measured climatic parameters. However, because of the procedural difficulty of synchronising recordings and processing of both temperature and relative humidity it is rather complicated. Therefore, it is advisable to use average annual values of temperature and relative humidity to calculate TOW (Tidblad et al., 2000).

Several efforts have been made by researchers to find a quantitative dependence of TOW on average annual and monthly temperature and relative humidity data (Barton et al., 1976; Sereda and Litvan, 1980; Lipfert et al., 1985; Kucera and Fitz, 1995). However, a clear and better understanding of these dependence became necessary in other to develop the relevant dose-response functions. Hence, Tidblad et al. (2000) came up with the following equation:

\[
TOW = 8766P(T)P(RH)
\]  
(6.3)

Where TOW= time of wetness (h/a), P(T)= normal probability distribution function for temperature, P(RH)= beta probability distribution function relative to relative humidity. Both P(T) and P(RH) range between 0<P<1, see Table 6.12.
Equation (6.3) is used along with the annual averages of temperature and relative humidity extracted from the UKCP09 database (Table 6.13) to estimate corrosion loss using the Klinesmith et al. (2007) model.

Table 6.11 Tabulated normal P(T) and beta P(RH) distributions for equation (6.2) for estimating TOW from average annual data on temperature and relative humidity (Tidblad et al., 2000)

<table>
<thead>
<tr>
<th>Rh, %</th>
<th>P(Rh)</th>
<th>T, °C</th>
<th>P(T)</th>
<th>Rh, %</th>
<th>P(Rh)</th>
<th>T, °C</th>
<th>P(T)</th>
</tr>
</thead>
<tbody>
<tr>
<td>20</td>
<td>0.002</td>
<td>−30</td>
<td>0.001</td>
<td>70</td>
<td>0.396</td>
<td>2</td>
<td>0.580</td>
</tr>
<tr>
<td>25</td>
<td>0.005</td>
<td>−28</td>
<td>0.002</td>
<td>72</td>
<td>0.438</td>
<td>4</td>
<td>0.656</td>
</tr>
<tr>
<td>30</td>
<td>0.012</td>
<td>−26</td>
<td>0.005</td>
<td>74</td>
<td>0.482</td>
<td>6</td>
<td>0.727</td>
</tr>
<tr>
<td>35</td>
<td>0.023</td>
<td>−24</td>
<td>0.008</td>
<td>76</td>
<td>0.527</td>
<td>8</td>
<td>0.789</td>
</tr>
<tr>
<td>40</td>
<td>0.042</td>
<td>−22</td>
<td>0.014</td>
<td>78</td>
<td>0.574</td>
<td>10</td>
<td>0.842</td>
</tr>
<tr>
<td>45</td>
<td>0.069</td>
<td>−20</td>
<td>0.022</td>
<td>80</td>
<td>0.622</td>
<td>12</td>
<td>0.886</td>
</tr>
<tr>
<td>50</td>
<td>0.107</td>
<td>−18</td>
<td>0.035</td>
<td>82</td>
<td>0.670</td>
<td>14</td>
<td>0.920</td>
</tr>
<tr>
<td>52</td>
<td>0.125</td>
<td>−16</td>
<td>0.054</td>
<td>84</td>
<td>0.719</td>
<td>16</td>
<td>0.946</td>
</tr>
<tr>
<td>54</td>
<td>0.146</td>
<td>−14</td>
<td>0.080</td>
<td>86</td>
<td>0.767</td>
<td>18</td>
<td>0.965</td>
</tr>
<tr>
<td>56</td>
<td>0.169</td>
<td>−12</td>
<td>0.114</td>
<td>88</td>
<td>0.814</td>
<td>20</td>
<td>0.978</td>
</tr>
<tr>
<td>58</td>
<td>0.194</td>
<td>−10</td>
<td>0.158</td>
<td>90</td>
<td>0.859</td>
<td>22</td>
<td>0.986</td>
</tr>
<tr>
<td>60</td>
<td>0.222</td>
<td>−8</td>
<td>0.211</td>
<td>92</td>
<td>0.900</td>
<td>24</td>
<td>0.992</td>
</tr>
<tr>
<td>62</td>
<td>0.252</td>
<td>−6</td>
<td>0.273</td>
<td>94</td>
<td>0.937</td>
<td>26</td>
<td>0.995</td>
</tr>
<tr>
<td>64</td>
<td>0.285</td>
<td>−4</td>
<td>0.344</td>
<td>96</td>
<td>0.968</td>
<td>28</td>
<td>0.998</td>
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<tr>
<td>66</td>
<td>0.320</td>
<td>−2</td>
<td>0.420</td>
<td>98</td>
<td>0.990</td>
<td>30</td>
<td>0.999</td>
</tr>
<tr>
<td>68</td>
<td>0.357</td>
<td>0</td>
<td>0.500</td>
<td>99</td>
<td>0.997</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

6.4 Quantitative assessment of corrosion loss based on different emission scenarios

Following definition of what the emission scenarios look like in the IPCC SRES and the UKCP09, it can be deduced that the definition probably fits in with the BS EN ISO 9223:2012 definition of the probable annual average ranges, geographical situation and deposition or concentration levels of the relevant environmental parameters.

Some of the emission scenario pathways refer to an expected rapid growing economy driven by the usage of fossil fuel which may later be substituted by renewable energy sources. These scenarios envisage a population growth of about 9 billion peaking at 2050’s. This description can be fitted into the P2 industrial atmospheric level for SO2 concentration with airborne salinity level of S2 and a TOW level of τ4 defining the temperature-relative humidity complex
of an open air or outdoor atmosphere in all climates (Tables 3, B.1, and B.3 & B.4 in ISO 9223:2012).

Based on these correlational assumptions the following parametric ranges were selected from the referred Tables in BS EN ISO 9223:2012.

- \( \text{SO}_2 \) deposition: \( 24 < P_d \leq 80 \text{ mg/m}^2/\text{day} \)
- Cl\(^-\) deposition: \( 60 < S_d \leq 300 \text{ mg/m}^2/\text{day} \)
- TOW: \( 2500 < \tau < 5500 \text{ h/a} \)
- T and RH: refer to Table 6.13

The selected ranges of the environmental parameters fall within the measured intervals of the parameters used in the derivation of the dose-response functions (Table 3 of ISO 9223:2012). Therefore, there is no risk of erroneous outcomes as the case may be when extrapolating the equations outside the intervals of the environmental parameters.
Table 6.12 Annual average temperature and relative humidity for Guildford, London-city, London extracted from UKCP09 user interface (average of 10,000 data each)

<table>
<thead>
<tr>
<th>30-yr period</th>
<th>Span</th>
<th>Temperature °C</th>
<th>Relative humidity %</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>low</td>
<td>medium</td>
</tr>
<tr>
<td>2020s</td>
<td>2010-2039</td>
<td>11.17</td>
<td>11.13</td>
</tr>
<tr>
<td>2030s</td>
<td>2020-2049</td>
<td>11.48</td>
<td>11.48</td>
</tr>
<tr>
<td>2040s</td>
<td>2030-2059</td>
<td>11.75</td>
<td>11.84</td>
</tr>
<tr>
<td>2050s</td>
<td>2040-2069</td>
<td>11.95</td>
<td>12.27</td>
</tr>
<tr>
<td>2060s</td>
<td>2050-2079</td>
<td>12.23</td>
<td>12.63</td>
</tr>
<tr>
<td>2070s</td>
<td>2060-2089</td>
<td>12.41</td>
<td>12.97</td>
</tr>
<tr>
<td>2080s</td>
<td>2070-2099</td>
<td>12.55</td>
<td>13.30</td>
</tr>
</tbody>
</table>

The ISO 9223 and 9224: 2012 models only capture the relevant environmental parameters within the first year of the corrosion phenomena as is indicated by equation (2.13). After that period, the potential effects of the changing environmental conditions are not reflected in the degradation process of the structural asset as can be seen in equation (2.36). This is a drawback of the ISO models. This research seeks to use a novel approach to capture the potential changes in the environmental parameters over time, following Kallias et al. (2016)’s proposals. Kallias et al. (2016) recasted equation (2.38) to capture the changing environmental conditions as follows:

\[ D(t < 20) = r_{corr}(20^b + b(20^{b-1})(t - 20)) \]  \hspace{1cm} (6.4)

\[ = r_{corr}20^b + r_{corr}b(20^{b-1})(t - 20) \]  \hspace{1cm} (6.5)
Looking at the equation above, the first part of the equation represents the damage within the first 20 years while the second part represents the damage beyond 20 years. Rearranging, Kallias et al. (2016) mathematically expresses the equation as follows:

\[ D(t > 20) = D(t = 20) + \frac{dD(t)}{dt}(t - 20) \]  

(6.6)

\[ = D(t = 20) + \sum_{i=1}^{n} \frac{dD(t)}{dt} T_i \]  

(6.7)

From the expression, it is obvious that the first part of the equation is exponential in nature while the second part is linear with a slope equal to

\[ \frac{dD(t)}{dt} = r_{corr} b(20^b-1) \]  

(6.8)

In the event of a change in exposure condition at a time (t) greater than 20 years, the slope of the linear portion can be modified accordingly by defining a new \( r_{corr} \) parameter to account for the new exposure conditions (Kallias et al., 2016). Therefore, for \( n \) changes in environmental conditions beyond 20 years equation (6.6) should be used. Note that \( T_i \) is the duration over which each \( r_{corr} \) is valid after the initial 20 years period.

Figure (6.4) shows the prediction of corrosion loss using the Kallias et al. (2016) model (equations (6.7)) and (6.8) for the three different emission scenarios based on the parametric assumptions discussed previously. The asset location was assumed to be away from the coast therefore, Cl\(^-\) was kept at 65mg/m\(^2\)/day for all scenarios while SO\(_2\) for the different scenarios was ranged as follows: 30-42mg/m\(^2\)/day, 43-61mg/m\(^2\)/day and 62-80mg/m\(^2\)/day for the low, medium and high emission scenarios respectively so as to accommodate the potential changes in the industrial atmospheric assumption earlier.
Comparing Figure 6.4 with Figures 6.5 to 6.10 which are obtained based on the original ISO models and the Klinesmith et al. (2007) model, it can be noticed that the ISO models underestimate corrosion loss. This is attributed to the fact that the ISO models are not able to capture the gradual changes in the exposure climate over time. At best, the whole life span of an asset is assumed to be captured by one climatic condition, which may not be the case considering climate change. For example, while the novel Kallias et al. (2016) model under low emissions scenario predicts corrosion loss of 7.0mm for a 100-year period, the ISO and Klinesmith et al. (2007) models predicted 0.91mm and 4.75mm respectively for the same time period using the 2020s environmental conditions. When the 2040s and 2060s 30-year future time period projections were used for the same low emission scenario, the ISO models indicated a section loss of 0.91mm and 0.83mm showing a 9% decline in damage for time periods expected to be even more aggressive. For the same 30-year time periods low emission, the Klinesmith et al. (2007) model turn out a damage of 5.15mm and 5.53mm. Which show a 7% increase in damage with time with progressive aggressive environment as will be expected. Again, the medium emission scenario damage assessment for the Kallias et al. (2016) model showed a section loss of 7.83mm at the same 100 years but, the ISO model
show a section loss of 1.17mm, 1.14mm and 1.11mm for the same 100 years for the 2020s, 2040s and 2060s 30-year future time periods projections, while the Klinesmith et al. (2007) model indicated a 5.40mm, 5.67mm and 6.52mm. Furthermore, the high emission scenario damage value for the novel Kallias et al. (2016) model is 8.67mm while the ISO model indicated 2.12mm, 2.04mm and 1.98mm for the 30-year future time periods defined above. For the Klinesmith et al. (2007) model a section loss of 6.23mm, 6.65 and 7.47mm are observed under the high emission scenario. It is evident that, one consistent trend with the ISO model values is that with future aggressive environmental projections a decrease is observed in damage while the Kallias et al. (2016) and Klinesmith et al. (2007) models were consistent in showing increases in damage with time because of the potential increase in climatic variables.

Figure 6.5 Corrosion loss in the different emission scenarios based on the ISO model and first year environmental condition in the 2020s
Figure 6.6 Corrosion loss in the different emission scenarios based on the ISO model and first year environmental condition in the 2040s

Figure 6.7 Corrosion loss in the different emission scenarios based on the ISO model and first year environmental condition in the 2060s
Figures 6.8 to 6.10 show the prediction of corrosion loss using the Klinesmith *et al.* (2007) model (equation (6.2)) for the three different emission scenarios. All the input data that has been used with this model is the same to the ones used in the ISO models above.

**Figure 6.8 Comparison of corrosion loss of the different emission scenarios for the 2020s using the Klinesmith *et al.* (2007) model**
For all corrosion predictions, it can be seen that the emission scenario considered affects the projected corrosion with the high emissions scenario resulting in the highest corrosion predictions. The ISO model shows that the corrosion prediction does not significantly change with different initial environmental conditions between the 2020s - 2060s (Figures 6.5 to 6.7).
This demonstrates that the dominating parameter in this model is the atmospheric pollution concentration, which has been kept constant over time in these figures.

The differences in the corrosion predictions obtained by the Klinesmith et al. (2007) model (Figures 6.8 to 6.10) between the different environmental condition decades (2020s to 2060s) appear to slight higher than the differences obtained from the ISO model.

**6.5 Impact of Climate Change on Buckling Strength**

Following the quantitative assessment of corrosion loss based on the different dose-response functions and emission scenarios, these predictions are then used to estimate the buckling strength loss of the steel plates over time, considering the effects of climate change. Since a full performance mapping as a result of the corrosion section loss has been presented in the previous two chapters, the corrosion loss predictions obtained in this chapter are converted into percentage thickness or volume losses and the buckling strength loss for these projections is obtained directly from the figures in Chapters 4 and 5 under compression and shear respectively. In order to illustrate the usefulness and potential capability of the reduction factor and the corrosion damage plots and how they can be used to assess the remaining critical and ultimate buckling strengths of plate elements as the suffer deterioration over time, Figures 6.11 to 6.35 were produced to pictorially demonstrate that. Tables C.1 - C.7 in appendix C show the data from which the referred figures and those following are produced from. It should however be noted that, the Figures presented are just a few representative cases of the enormous potential capability of these reduction factor and deterioration plots. They reduction factors cover both compressive and shear load cases for two aspect ratios.
Figure 6.11 Compressive strength remaining factors based on (ISO 9223 & 9224:2012) models & uniform corrosion, AR=1.23, low emission scenario 2060’s

Figure 6.12 Compressive strength remaining factors based on (ISO 9223 & 9224:2012) models & uniform corrosion, AR=1.23, medium emission scenario 2060’s
Figure 6.13 Compressive strength remaining factors based on (ISO 9223 & 9224:2012) models & uniform corrosion, AR=1.23, high emission scenario 2060’s

Figure 6.14 Compressive strength remaining factors based on (Klinesmith et al. (2007) model & uniform corrosion, AR=1.23, low emission scenario 2060’s
Figure 6.15 Compressive strength remaining factors based on (Klinesmith et al. (2007) model & uniform corrosion, AR=1.23, medium emission scenario 2060’s

Figure 6.16 Compressive strength remaining factors based on (Klinesmith et al. (2007) model & uniform corrosion, AR=1.23, high emission scenario 2060’s
Figure 6.17 Compressive strength remaining factors based on (Kallias et al. 2016) model & uniform corrosion, AR=1.23, low emission scenario 2060's

Figure 6.18 Compressive strength remaining factors based on (Kallias et al. 2016) model & uniform corrosion, AR=1.23, medium emission scenario 2060’s
6.5.1 Reduction in critical strength under uniform corrosion

Figures 6.11 to 6.19 show that the critical buckling strengths of the plates under compressive load and uniform corrosion deterioration degrades over time. For example, Figures 6.11 to 6.13 representing the ISO 9223 & 9224: 2012 dose response models estimation for low, medium and high emission scenarios indicated that from an as-new nonlinear critical strength of 85.7N/mm$^2$, the remaining plate capacity at 100 years under uniform corrosion is 71.13N/mm$^2$, 67.70N/mm$^2$ and 60.85N/mm$^2$ in the low, medium and high emission scenarios respectively. This show a capacity loss of 17%, 21% and 29% respectively for the plate with aspect ratio (AR) equal to 1.23. For the Klinesmith model, remaining capacity of 25.71N/mm$^2$, 16.28N/mm$^2$ and 11.14N/mm$^2$ is observed for the same conditions with the ISO, showing a percentage strength loss of 70%, 81% and 87% respectively in 100 years. The Kallias model indicated a remaining capacity of 16.28N/mm$^2$, 11.14N/mm$^2$ and 8.57N/mm$^2$ meaning 81%-87% strength loss in 100 years is possible. The remaining capacity of the plate at any given time can also be read off from the plots as demonstrated above. The Klinesmith models Figures 6.14 to 6.16 showed strength reduction from the start unlike the Kallias Figures 6.17 to 6.19 whose strength reduction became more pronounced around 20 years in the life of the asset. The somewhat gradual reduction in Figures 6.17 to 6.19 for the Kallias
model is indicative of gradual strength loss within the first 20 years consistent with the notion of reduced deterioration as a result of corrosion product accumulation on metal surface.

**6.5.2 Reduction in ultimate strength under uniform corrosion**

Just like the critical strength is observed to degrade over time with corrosion effect, ultimate strength is also noticed to drop. Figures 6.11 to 6.19 demonstrate that also. For example, at 100 years in the life of the asset the ISO model show that 10%, 11% and 16% of ultimate strength will be lost in the low, medium and high emission scenarios. While the Klinesmith model show an ultimate strength loss of 43%, 50% and 57% in the low, medium and high emission scenarios. The Kallias model which is considered the most critical or worst case scenario indicates 50%, 56% and 61% ultimate strength loss respectively for the stated emission scenarios.

**6.5.3 Reduction in strength under non-uniform corrosion**

Having observed that the Kallias model estimates the worst case corrosion scenario, therefore, its deterioration estimations will be used further to demonstrate the reduction in strength for a few more corrosion patterns or configurations. Figures 6.20 to 6.22 show the reduction in strength for the non-uniform corrosion configuration 1 condition 1 based on the Kallias model estimation. In the low emission scenario at 100 years the Kallias model projected 84% remaining strength which is equal to 133.06N/mm² from the as-new strength of 158.4N/mm². The medium and high emission scenarios projected 75% and 64% remaining strength equalling 118.80N/mm² and 101.38N/mm² respectively. These projections are indicating a drop in ultimate strength of 16%, 25% and 36% for the low, medium and high emission scenarios. From the plots the same can be read in terms of critical strength.
Figure 6.20 Compressive strength remaining factors based on (Kallias et al. 2016) model & non-uniform corrosion configuration 1, AR=1.23, low emission scenario 2060’s

Figure 6.21 Compressive strength remaining factors based on (Kallias et al. 2016) model & non-uniform corrosion configuration 1, AR=1.23, medium emission scenario 2060’s
A condensed reduction in strength plots comparing low, medium and high emission scenarios of remaining ultimate strength versus time (years) is herewith presented Figures 6.23 to 6.27 to demonstrate in another fashion the reduced strength behaviour of the plates under compressive stress action overtime.

**Figure 6.22 Compressive strength remaining factors based on (Kallias et al. 2016) model & non-uniform corrosion configuration 1, AR=1.23, high emission scenario 2060’s**

**Figure 6.23 Ultimate strength reduction over time based on ISO 9223 & 9224:2012 uniform corrosion**
Figure 6.24 Ultimate strength reduction over time based on (Klinesmith et al. 2006) uniform corrosion

Figure 6.25 Ultimate strength reduction over time based on (Kallias et al. 2016) uniform corrosion
Figure 6.26 Ultimate strength reduction over time based on (Kallias et al. 2016) non-uniform corrosion configuration 1 condition 1(0.1b)

Figure 6.27 Ultimate strength reduction over time based on (Kallias et al. 2016) non-uniform corrosion configuration 1 condition 3(0.5b)
6.5.4 Reduction in strength under shear load uniform corrosion

Figures 6.28 to 6.30 show the degradation of linear critical and ultimate buckling strength under shear load and uniform corrosion using the Kallias model as the worst case scenario. They cases presented are for the 1.23 (AR) plate, the remaining ultimate strength in the low, medium and high emission scenarios is 85.56N/mm², 75.29N/mm² and 63.31N/mm² respectively, representing 44%, 51% and 59% strength loss in 100 years based on 154.0N/mm² as-new plate ultimate strength for the 1.23 AR plate. Furthermore, the first twenty years show a gradual and steady strength loss as can be noticed by the plateau exhibited, but afterwards a sharp drop in strength is observed (Figures 6.28 to 6.30).

Figure 6.28 Shear strength remaining factors based on (Kallias et al. 2016) model & uniform corrosion, AR=1.23, low emission scenario 2060’s
Figure 6.29 Shear strength remaining factors based on (Kallias et al. 2016) model & uniform corrosion, AR=1.23, medium emission scenario 2060’s

Figure 6.30 Shear strength remaining factors based on (Kallias et al. 2016) model & uniform corrosion, AR=1.23, high emission scenario 2060’s
Figure 6.31 Comparative ultimate strength reduction based on (Kallias et al. 2016) model & uniform corrosion configuration 1 condition 3, AR=1.23, for the three emission scenarios 2060’s

6.5.5 Reduction in shear strength non-uniform corrosion

The non-uniform corrosion situations are illustrated in Figures 6.32 to 6.35. It is observed that, as generally seen throughout, the emission pathway plays a key role in the degradation degree. For example, generally, the high emission scenario show greater damage to strength over time. At 100 years projections it is observed that in the low emission category ultimate strength is lost by 47%, while at the medium and high category 57% and 63% strength is lost. Though within the first 35 years the strength loss behavioural patterns seem similar Figure 6.35.
Figure 6.32 Shear strength remaining factors based on (Kallias et al. 2016) model & non-uniform corrosion configuration 1 condition 3, AR=1.23, low emission scenario 2060’s

Figure 6.33 Shear strength remaining factors based on (Kallias et al. 2016) model & non-uniform corrosion configuration 1 condition 3, AR=1.23, medium emission scenario 2060’s
Figure 6.34 Shear strength remaining factors based on (Kallias et al. 2016) model & non-uniform corrosion configuration 1 condition 3, AR=1.23, high emission scenario 2060’s

Figure 6.35 Comparative ultimate strength reduction based on (Kallias et al. 2016) model & non-uniform corrosion configuration 1 condition 3, AR=1.23, for the three emission scenarios 2060’s
The importance and implication of performance prediction to an asset owner and manager is very critical from the perspective of financial constraints in terms of designing new assets, where the potential effects of corrosion over the service life of the asset need to be assessed. However, it is also very important from the point of view of existing assets with regards to maintenance, repair and replacement of components. The buckling performance plots developed in this chapter for the different scenarios can assist asset owners to quickly assess the implications of different corrosion intensities and extents on performance. This can be useful, for example, following inspections where corrosion of plate elements has been detected or in cases where managers require to carry out whole-life analysis to determine maintenance strategies in the long-term.

6.6 Concluding remarks
This chapter from the beginning set out to quantify corrosion loss based on the potential changes in the exposure condition of an asset. This is carried out in order to link climate change to structural integrity with respect to buckling strength, which is a key novelty of this research. The dose-response functions that link the corrosion damage to the relevant environmental variables were used. It was found that, although robust and universally applicable, the ISO model tend to underestimate the corrosion damage which is a serious limitation from the perspective of infrastructural management. On the other hand, the models proposed by Kallias et al. (2016) and Klinesmith et al. (2007) were found to result in comparable predictions for corrosion loss over time. The reason for the low corrosion damage assessment predicted by the ISO models is the fact that they do not consider and capture the potential changes in the climate over time as do the other two models. The corrosion losses were further extended into buckling performance modelling where the results clearly show that climate change can have a significant degrading effect on the buckling strength of plate elements, depending on the emission scenarios. It is evident that, corrosion loss type and pattern plays a role in the weakening of strength as well emission scenario.
CHAPTER SEVEN

Summary, Conclusions and Recommendations

7.1 Introduction
The aim of this chapter is to draw a broad and general conclusion on the already identified results from the investigations carried out. These results forms the key findings of this research. The conclusions to be drawn and recommendations to be made will be linked to the aim and objectives set out from the onset in chapter one. The influence of potential increases in current and future environmental parameters as a result of climate change and the main pollutants sulphur dioxide and chlorides on atmospheric corrosion of carbon steel as determined will be related to buckling performance of plate elements through normalised reduction factor plots. This is viewed to be a very useful early design decision tool as well as an assessment tool for infrastructure managers in planning maintenance and repairs in the face of uncertainties and budgetary constraints. At the end of the chapter a number of suggestions are made for future work.

7.2 Summary
In this research, a novel corrosion estimation dose-response function is utilised to address the seeming limitation of the BS EN ISO 9223 & 9224 (2012) models as well as the shortfall of the Klinesmith et al. (2007) model. This novel model wisely captures the potential changes in climate parameters over the lifespan of the structural asset which is not so with the ISO model in particular. Therefore, the corrosion damage is reasonably assessed with the novel model such that the degradation of the critical and ultimate buckling strength in terms of both
compressive and shear stresses can be predicted across the life of the infrastructural asset. This capability of assessing the buckling strength degradation can prove very useful in terms of both performance and condition assessment of metallic bridge members on a road or rail network.

Consistent with this research aim and objectives of investigating the effects of climate change on the buckling strength of plate elements with the view to improving lifetime structural performance of metallic bridges, the following objectives were achieved:

Parametric appraisal and use of the changes in atmospheric environmental parameters and pollutants which define the corrosivity environment of an asset for long-term corrosion predictions for structural steel.

The long-term prediction of corrosion damage is evaluated based on the data from the three defined IPCC emission scenarios (low, medium and high) accessed from the UKCP09 user interface. These climatic data was utilised through the consideration of three different dose-response functions: BS EN ISO 9223 & 9224 (2012), Klinesmith *et al.* (2007) and the novel Kallias *et al.* (2016) modified ISO model. The results showed that the novel model best assesses the damage in the light of the present reality of global climate change to which the infrastructural asset is exposed to.

Another objective of this research was the performance assessment of metallic plate elements used in bridge structures made of carbon steel under climate change variability. To achieve this, complete spectral mapping of plate elements under different corrosion patterns and deterioration conditions was carried out using linear and nonlinear finite element analysis with the ABAQUS commercial package. A total of 522 simulations were carried out on plates with two aspect ratios, i.e. 1.23 and 2 where the critical and ultimate buckling strength of each plate was determined and then further normalised and plotted in to reduction factor curves. The plates were analysed under two load case scenarios, compressive and shear load cases. The nonlinear finite element analysis was considered to account for both geometric and material nonlinearities. With these reduction factor curves under full ranges of corrosion from intact plates down to entire thickness loss, the damage assessed using the dose-response functions can be read off and the capacity of the member can be easily determined. For example, based on the Kallias *et al.* (2016) prediction a corrosion damage of 6.44mm in 100 years under low emission scenario is possible. The predicted damage is equivalent to 51%
section thickness loss in uniform corrosion for a 12.7mm thick plate. If considering uniform corrosion, the uniform corrosion reduction factor curve should be used to read off reduction in strength as a result of the 51% thickness loss due to climate change variability. 

The finite element models developed and used for this research are novel and were developed to represent relatively corrosion patterns observed on bridge plate girders. They patterns modelled are the uniform and non-uniform corrosion. The non-uniform was further divided into two configurations (1 & 2). Configuration 1 represented a scenario where only the bottom portion of the plate element is wasted and configuration 2 represented a scenario where both left and right edges of the plate along with the plate bottom are wasted by corrosion. However, for the purpose of simulation convergence the patterns were idealised; the results obtained from the models were generally in good agreement with the theoretical values as computed from the well-known Euler buckling equation. For the compressive load cases the differences between the finite element outcome and theory were relatively in good agreement (about 4%). For the shear load scenarios, the differences were between 6% & 7% owing to challenges in simulating the exact boundary conditions. This difficulty of boundary condition simulation affects the value of the shear buckling coefficient.

This way the objective of proposing finite element models for strength assessment of deteriorated steel plate elements was achieved.

In order to enable continuous and regular monitoring and management of metallic bridge elements with respect to long-term environmental deterioration capturing the potential effects of climate change, this research study provides a pathway for such performance assessment over time, fulfilling the fifth objective. The research outcomes delivered can prove useful to asset owners to quickly assess the performance implications of different levels and extents of corrosion on steel plate elements after inspection. This can feed into whole-life analysis models where long-term performance modelling is an important consideration as well as assess the criticality of different levels of corrosion, potentially detected through bridge inspections. The results can also be helpful towards determining bridge capacity ratings and allowable loads.

7.3 Research conclusions
The main conclusions of the present work, can be summarised as follows:
Climate Change was found to be capable of significantly affecting the degradation of the buckling performance of corroded plate elements, depending on the scenario pathway considered for the assessment.

The existing dose-response function presented in the BS EN ISO 9223 and 9224:2012 were found to underestimate corrosion predictions as compared with the Klinesmith et al. (2007) and the Kallias et al. (2016) models, which are able to capture potential changes in environmental variables and atmospheric pollutant concentrations.

Finite element analysis was found to be a promising technique to support the analysis of structural plate elements under different corrosion scenarios, as the initial benchmark and verifications studies showed relatively small differences between numerical and analytical predictions.

From the damage computation based on projected climatic data from the UKCP09 database using the Kallias et al. (2016) model, it was found that under the low emission scenario and for time horizon of 100 years, up to 51% corrosion loss damage can result. On the other hand, the medium and high emission scenarios indicated 58% and 65% damage for a 100 year period.

The uniform corrosion scenario results showed that buckling strength reduction becomes critical from 30% thickness loss upwards, for which between 40% to 99% of the compressive critical and ultimate buckling strength is lost (for both aspect ratios AR=1.23 & 2 plates).

Under the non-uniform corrosion and both configurations (1 & 2) scenarios, it is observed that the following patterns are the worst case scenarios: 90% -10%, 70% -30% and 50% -50% thickness-depth combination patterns downwards. For these cases, a range of between 56% to 99% compressive strength is lost.

The shear load case uniform corrosion patterns show criticality from 50% thickness loss upwards, for which 75% to 99% of strength is lost for both aspect ratio plates. The non-uniform corrosion scenarios show criticality of wastage from the following thickness-depth loss amounts: 70% -10%, 70% -30% and 50% -50% where between 56% to 97% strength is lost for both ARs.

7.4 Recommendations
Based on the achieved outcomes, the following recommendations can be drawn:
That designers and infrastructural managers should consider adopting for use deterioration models that can take into account the potential changes in environmental and atmospheric pollution variables, for long-term corrosion predictions. The use of the Kallias et al. (2016) assessment approach is recommended to quantify the expected damage since it can capture gradual changes over time in these variables and it also provides an upper bound prediction as compared with other similar models, i.e. the ISO and Klinesmith et al. (2007) models. Since all these state-of-the art models are a function of environmental parameters (i.e. temperature, relative humidity, time of wetness) and atmospheric pollutant concentrations (i.e. sulphur dioxide, chlorides, etc.), with the availability of more detailed data on these parameters through monitoring they have a great potential towards providing more reliable long-term corrosion estimates.

For the design of new structures, the results of this research can be used to consider in advance long-term deterioration estimates at the design stage so that preventive maintenance strategies can be considered at an early stage. For example, thickness allowance or tolerance provision against corrosion can be influenced positively by the use of the reduction factor plots proposed by this research. If a designer is able to project the expected changes in the relevant climatic and atmospheric variables defining the environment of existence of the new proposed structure, the designer can quantify the expected damage using the novel dose-response function and then quantify the expected degradation on buckling capacity over the design lifespan of the structure. This will certainly contribute towards more efficient life-cycle planning of new assets considering these performance models throughout the whole-life of an asset.

For existing structures that they may already have suffered from a combination of damage factors ranging from aging, aggressive environments, deterioration from service demands of increasing traffic and heavier loads and other physical mechanisms coupled with deferred maintenance as a result of the budgetary constraints. The results from this research may serve as a valuable tool to guarantee a more responsive continuous and regular monitoring of infrastructure assets, supporting proper on time strategic maintenance planning. The results can provide a quick way of assessing the implications of corrosion detected from bridge inspections by quantifying the remaining strength of the elements through the strength reduction plots produced in
this research, leading towards more efficient decisions on the maintenance actions to be carried out. Furthermore, these research outcomes, given their capability to be used for performance assessment, can be utilised for the purposes of bridge rating, where necessary, in the interest of both structural and public health in line with relevant bridge rating frameworks.

- Owing to the magnitude of the damages observed within the lifetime of the infrastructural asset, the outcomes of this research can be a helpful guide in influencing the engineering selection of bridge material given infrastructural asset environmental exposure in the current reality of climate change.

7.5 Recommendations for future work

In the course of the research carried out in this thesis, a few topics worth further investigation have been identified:

- This work can be further extended into structural reliability analysis in order to enable a more reliable estimation of the failure probability for metallic bridge structural design situations. It is obvious that climate change variability is full of uncertainties and some available information may not even be complete, hence, the need for the reliability analysis to rationally treat these uncertainties.

- Comparison of the predictions provided by the deterioration model with field measurements of real corrosion amounts from bridge assets can provide useful insight towards the confidence of the predictions. Data obtained from inspection reports combined with environmental data can be utilised towards that purpose.

- Obviously, this research has concentrated on the buckling resistance of plate elements. Extending the performance modelling in other behaviour types, such as fatigue from cyclic loading combined with long-term corrosion can lead into additional performance models to capture these.

- In the light of environmental assessment of bridge material choices and design, this work can be extended to look at whole-life analysis considering the costs with respect to different maintenance strategies to identify in more detail the optimum sequence of maintenance activities to be carried out during the service life of assets.