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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

A QUANTITATIVE INVESTIGATION OF THE CARBON SAVINGS IN STRUCTURAL DESIGN BASED ON THE EVOLUTION OF THE DESIGN CODES

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Declaration

I, Romana Gina Katrakilis, declare this research project as my unaided work. All the work in this document, save for that which is appropriately acknowledged, is my own.
It is being submitted for a MASTER'S degree at the University of the Witwatersrand. It has not been submitted before for any degree or examination in any other University.

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ABSTRACT

There have been several iterations of structural building codes over the past decades, with varying structural design methodologies. The unintentional effect that the evolution of the structural design codes had on the carbon dioxide emissions of structural elements was investigated in this research report. As the effects of climate change become more prevalent, such as increased surface temperatures which lead to extreme weather events, understanding the unintentional carbon dioxide reductions in structural design is essential to meeting the greenhouse gas emission goals from the construction sector, such as the Sustainable Development Goals (SDGs) set out by the United Nations. This research investigates the difference in the carbon dioxide emissions, measured in kgCO2-equivalents, that reinforced concrete beams and columns generate when designed using the Code of Practice CP114 (1960s) and the Eurocodes (2010) structural design methodologies.

Concrete is a widely used engineered material due to its many advantages, such as strength, versatility, workability, abundant and cheaply accessible primary materials and relatively cheap manufacturing processes. However, cement production emits a significant quantity of carbon dioxide and other greenhouse gases. In the construction sector, a primary method of reducing emissions is by reducing the required quantities of materials. A reduction in the concrete quantity required for an element to perform its intended function can lead to a reduction in the embodied carbon of the element, provided that a large volume of reinforcement is not required to sustain the loads. Embodied carbon is the amount of greenhouse gases emitted during all stages of a product's life, including extraction, transport, manufacture, and installation (EPA, 2023).

The global carbon dioxide emissions are continually being monitored by the United Nations and other organisations. This is done to accurately report on, and minimise, carbon dioxide emissions and the subsequent effects of climate change. Hence, it is necessary to investigate and report any carbon dioxide emissions reductions in the construction industry.

Two structural design codes were used to evaluate the embodied carbon of two structural elements, namely a simply supported beam and a column in a concrete frame office building, subject to permanent and imposed loading, as stipulated by the structural design codes. These structural design codes were in use more than 50 years apart. In Ireland, the Eurocodes are currently in use, whereas the British Standards Code of Practice CP114 was in use in the 1960s and 1970s.

The findings of this research are that across the designs of both elements, there was a decrease in the quantity of concrete and reinforcement required to sustain the design loads in the Eurocodes, which confirms the hypothesis of this research paper. This research project only investigated the effect of reduced material quantities on the embodied carbon of the elements. The effects of partially replacing the cement in the concrete with Supplementary Cementitious Materials (SCMs) was not considered.

Therefore, the evolution of the design codes led to an unintentional reduction in embodied carbon between the CP114 code and the Eurocodes.

1. INTRODUCTION

1.1. Background

The rise in surface temperatures over the past century has led to changes to the earth's climate, known as climate change. Climate change refers to changes in the Earth’s temperature and weather patterns. These are primarily driven by human activities, such as the burning of fossil fuels and other activities that generate greenhouse gas emissions. The greenhouse gases emitted in these activities, such as carbon dioxide, act as a blanket over the Earth, trapping heat and subsequently raising temperatures (IPCC, 2022). Increased surface temperatures lead to extreme weather events, such as severe drought, stronger storms, and frequent flooding. The hotter temperatures can also lead to accelerated ice melting, resulting in rising sea levels (The Climate Reality Project, 2023).

Due to climate change, it has become increasingly important to minimise carbon dioxide and other greenhouse gases from entering the atmosphere, which would trap more heat, and cause further temperature rises. This is particularly true for the construction industry, which contributes to global annual carbon emissions. The 2022 Global Status Report for Buildings and Construction found that the construction sector accounts for approximately 37% of energy-related carbon dioxide emissions (UNEP, 2022).

Concrete is one of the most widely used engineered materials on Earth due to its many advantages in the construction sector, such as strength, versatility, workability, abundant and cheaply accessible primary materials and relatively cheap manufacturing processes. However, the production of cement, which is used in concrete, is a carbon-intensive process. This is due to the carbon dioxide that is released from the chemical reactions in the production of cement clinker, the energy required to heat the rotary kilns in the manufacturing process (which usually comes from the burning of fossil fuels). Furthermore, the transport of materials, such as fine and coarse aggregate, which are components used in concrete design mixtures, contribute to the concrete’s carbon footprint.

Concrete mix designs and the design of reinforced concrete structures are governed by structural design codes. These national codes, or standards, inform the design methodology, concrete constituents, structural loads, concrete and reinforcement strength, etc. Material standards are also used to inform material qualities.

This research project aims to determine the extent to which structural design codes assist in the quest for sustainable infrastructure. If a structural designer were to rely solely on design codes, without relying on any inventive sustainable technology, would the building unintentionally achieve any embodied carbon savings compared to its historic counterpart?

The evolution of the structural design codes between a period of 50 years was evaluated. This led to a better understanding of the extent to which the current codes are promoting the design of structural members with optimised dimensions, leading to less carbon-intensive structures. This was investigated by comparing the design calculations and design coefficients for two structural elements: a beam and a column. The structural elements were designed using the current codes in Europe (as this is where the student is based), and the historic codes, to compare the embodied carbon of each design.

Embodied carbon is a measure of the amount of greenhouse gases emitted during all stages of a product's life, including extraction, transport, manufacture, installation, etc. The boundary of the life cycle assessment (LCA) indicates which stages of the products life are under assessment (EPA, 2023). Embodied carbon is measured in kilograms of CO_2-equivalents ($kgCO_2$-e). This metric is used to compare the emissions from different greenhouse gases in relation to their global warming potential (GWP) (Eurostat, 2017). Therefore, the embodied carbon of a structural element is a function of the volume of materials used, as well as the type of material and its properties, over the specified LCA boundary. The boundaries of this research project were cradle to gate.

The beams and columns were designed using the current and the historic structural design codes. Different reinforced concrete sections were generated from each of the codes due to the differences in the analysis procedures specified in each of them. Therefore, using a similar assumed embodied carbon for each of the concrete design mixes, any changes in the carbon dioxide emissions were derived directly from the optimization of the dimensions of the section of the member, and the required reinforcement to sustain the load.

An optimized reinforced concrete section ultimately has less concrete required. However, this does not necessarily correlate to a reduction in reinforcement, as more bars may be required to sustain the proposed load. Optimized structural members require smaller quantities of materials to safely perform their intended functions. For example, an efficiently designed ground slab would be thinner than its over-designed counterpart, which leads to less concrete required for the slab. A reduction in the concrete required for an element to perform its intended function can lead to a reduction in the embodied carbon of the element, provided that a large volume of reinforcement is not required to sustain the loads. This study was used to determine the carbon dioxide savings purely from the optimization of the element dimensions and the reductions in material quantities, without considering the quality and composition of those materials.

1.2. Aim

This research aims to establish whether structural design codes have evolved over a period of more than 50 years to bring about any unintended yet significant embodied carbon dioxide emissions reductions, considering a cradle to gate Life Cycle Assessment (LCA) boundary.

First, it should be established whether the structural member’s cross-sectional dimensions and reinforcement quantities have been optimised and reduced between standards published 50 years apart, as a direct result of the changes to the design methodologies and procedures in the codes. If so, a comparison of the materials required for each design should be calculated, and the corresponding reductions in embodied carbon quantified for each of the designs.

1.3. Hypothesis

The embodied carbon of a modern structural element designed using the Eurocodes (2010) is lower compared to the same element designed to the Code of Practice CP114 which was used over 50 years ago, due to optimised cross-sectional dimensions and the required quantity of reinforcement.

1.4. Objectives

The objectives of this study include:

- Identify the structural design codes used in practice today, as well as their predecessors, over 50 years ago.
- Compare the design philosophies of the structural design codes.
- Complete the design of a simply supported beam and a column set in a concrete frame office building in Ireland. Simply supported beams and columns are both commonly used structural elements in a building. These will be designed using the current structural design code and then using the historic structural design codes.
- Compare the cross-sectional dimensions and required reinforcement area for each corresponding design to establish whether these have been optimized over the period of 50-years.
- Compare the embodied carbon of the elements designed to the current and the historic codes based on the quantities of materials required in each case.

1.5. Scope and Limitations

1.5.1. Scope

There are several methods which can be used to reduce the embodied carbon of reinforced concrete structures, such as replacing a proportion of the cement with Supplementary Cementitious Materials (SCMs), optimizing and reducing the quantity of materials required for a member to perform its intended purpose, using pre-stressed reinforced concrete elements, etc.

This research project focused on the optimization and reduction of materials, if any, over the 50-year period. In other words, to establish whether the cross-sectional dimensions of a member, as well as its required reinforcement area, were optimized as the structural design codes evolved.

Two structural members, a beam and a column, were designed using two different structural design codes. One of the structural design codes is currently used in Europe, known as the Eurocodes. The other structural design code was in use more than 50 years ago, called the Code of Practice CP114. The evolution of these codes was evaluated through these designs by determining which code led to optimised cross-sectional dimensions and an optimised quantity of reinforcement required to sustain a given load. The embodied carbon of each design was then calculated, to determine whether any significant carbon dioxide savings have been established over 50 years, purely by optimising the dimensions of the element, and the amount of reinforcement required.

Different reinforced concrete sections were generated from each of the codes due to the differences in the analysis procedures specified in each of them. Therefore, using the same embodied carbon for each of the concrete design mixes, any changes in the carbon dioxide emissions were derived directly from the optimization of the dimensions of the section of the member, and the required reinforcement to sustain the load.

1.5.2. Limitations

This research project only focused on the structural design of a simply supported beam and a column. This was due to the time constraints of the master's program. Additionally, only two design codes were compared which were in use 50 years apart: the current Eurocodes and the historic Code of Practice CP114. However, there have been several iterations of design codes in the period between these two codes, such as Code of Practice CP110, which was in use from the early 1970s to the mid-1980s, which were not considered.

This research project only investigated the codes that are currently used, and which were historically used, in Ireland, as that is where the student is based. Therefore, the results are limited to the region where these codes were used. These were used in Ireland and the United Kingdom.

This study was used to determine the carbon dioxide savings purely from the optimization of the element dimensions and the reductions in material quantities, without considering the quality and composition of those materials. Therefore, SCMs were not introduced into the scope of this research.

An embodied carbon calculation software was used to determine the embodied carbon of the structural elements. This software takes multiple factors into account, such as the volume of concrete and reinforcement in the element, as well as the concrete mix design. To establish the embodied carbon changes based solely on the optimized material quantities, it was assumed that the concrete mix designs from both design codes were similar, taking into account only any explicit instructions from the codes with regards to the concrete mix design. This was done to ensure that the embodied carbon per given unit volume of concrete was the same.

This is a broad assumption which has its limitations. For example, the type of cement used could differ between the decades, as well as the usual percentage of SCM replacement. The required water content and cement content might differ. The composition of the concrete might differ more in terms of aggregate sizes, quality, and quantity. However, these factors have not been considered in this research.

1.6. Outline of this Research Project

This research project is comprised of six chapters.

Chapter 2 presents a literature review of cement, concrete, and reinforcement properties, life cycle assessments and embodied carbon. The evolution of structural design philosophies for the codes are also reviewed.

Chapter 3 describes the methodology used in this research project for the design of a simply supported beam and a column in accordance with the Eurocodes and Code of Practice CP114. The methodology for determining the embodied carbon of structural elements is also described.

Chapter 4 presents the results of the simply supported beam and column designs, as per the methodology in Chapter 3.

Chapter 5 presents the results of additional beams and columns which were investigated, taking different lengths, concrete strengths, and loads into account. The same methodologies from Chapter 3 were used.

Chapter 6 presents the discussion of the results determined in Chapter 4.

Chapter 7 presents the conclusion of this study. Based on the identified gaps and limitations, recommendations for further research is also noted.

2. LITERATURE REVIEW

2.1. Introduction

The rise in surface temperatures over the past century has led to changes to the earth's climate, known as climate change. Climate change refers to changes in the Earth's temperature and weather patterns. These are primarily driven by human activities driven by human activities, such as the burning of fossil fuels and other activities that generate greenhouse gas emissions. The greenhouse gases emitted in these activities, such as carbon dioxide, act as a blanket over the Earth, trapping heat and subsequently raising temperatures (IPCC, 2022). The consequence of these changes is the increased risk of extreme weather events such as severe drought, stronger storms, and frequent flooding. The hotter temperatures can also lead to accelerated ice melting, resulting in rising sea levels. (The Climate Reality Project, 2023). As a consequence, in the past few decades, it has become increasingly important to minimise carbon dioxide and other greenhouse gases from entering the atmosphere, which would trap more heat, and cause further temperature rises.

This is particularly true for the construction industry, which contributes to global annual carbon emissions. The 2022 Global Status Report for Buildings and Construction found that the construction sector accounts for approximately 37% of energy related carbon dioxide emissions (UNEP, 2022).

2.2. Climate Change

2.2.1. Atmospheric Carbon Dioxide Concentrations

Climate change is measurable, and this is done by monitoring climate change indicators. One of the climate change indicators which is monitored by global, international, and national organisations, such as the United States Environmental Protection Agency (U.S. EPA), is the Atmospheric Concentrations of Greenhouse Gases. This climate change indicator quantifies and describes the changes of the concentrations of major greenhouse gases in the Earth's atmosphere. Among the other greenhouse gases, such as methane and nitrous oxide, carbon dioxide concentrations are measured and monitored. The graph in Figure 1 shows the global concentrations of carbon dioxide in the atmosphere over time, measured in parts per million (ppm). This recent data is collected from air monitoring sites around the Earth, and the historic data was collected from ice core studies (U.S. EPA, 2024).

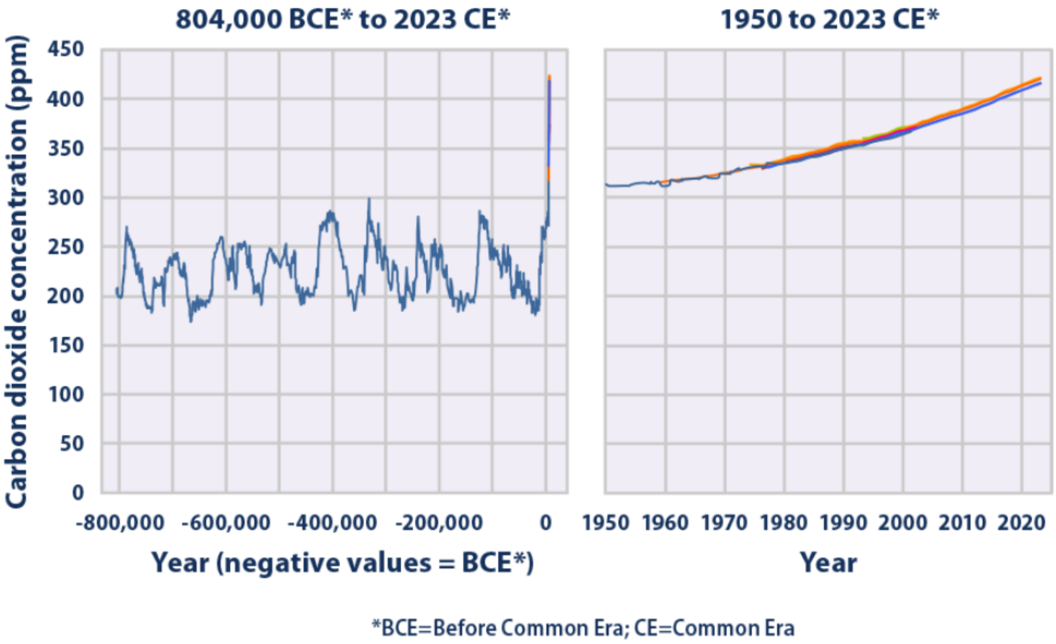

Figure 1: Global carbon dioxide concentrations in the atmosphere over time (U.S. EPA, 2024)

From this graph, it is evident that historically, carbon dioxide concentrations in the atmosphere have risen and fallen over hundreds of thousands of years. Greenhouse gases can remain in the atmosphere for a long time, either for decades or for millennia. Over time, they are removed from the atmosphere through chemical reactions or by being absorbed by the oceans and vegetation. Hence, the rise and fall of the carbon dioxide concentrations (U.S. EPA, 2024).

However, in the last few hundred years, the concentrations of carbon dioxide have risen significantly, shown by a sharp vertical spike in the graph, without any falls in the concentrations. The level of this spike is higher than any of the historic concentrations and is shown to be continually rising. According to the U.S. EPA, this is almost entirely due to human activities. Human activities cause large concentrations of greenhouse gases to enter the atmosphere more quickly than they can be removed (U.S. EPA, 2024).

The rise in carbon dioxide concentrations have been significant since the dawn of the industrial era, when the burning of fossil fuels and the rise of deforestation began. The concentrations rose from 280ppm in the late 1700's to 419ppm in 2023. This correlates to a 49% increase (U.S. EPA, 2024).

2.2.2. The Sustainable Development Agenda

In 2015, the United Nations (UN) introduced the 2030 Agenda for Sustainable Development, which sets out 17 interconnected goals. This agenda was adopted by all the countries in the UN. These 17 goals, which are described briefly below, are called the Sustainable Development Goals (SDGs), and aim to address worldwide critical and urgent issues, such as poverty, human health, equality, climate change, and biodiversity, amongst others. Whilst the Goals are a call for action to both developed and developing countries alike to combat climate change and to promote prosperity, they are not legally binding (United Nations, 2024).

- Goal 1: No poverty – end poverty in all forms (United Nations, 2024).
- Goal 2: Zero hunger – end hunger, achieve food security, improve nutrition, promote sustainable agriculture (United Nations, 2024).
- Goal 3: Good health and well-being – ensure and promote healthy lives and well-being at all ages (United Nations, 2024).
- Goal 4: Quality education – ensure inclusive and quality education and promote lifelong learning (United Nations, 2024).
- Goal 5: Gender equality – achieve gender equality and empower all women and girls.
- Goal 6: Clean water and sanitation – ensure availability and sustainable management of water and sanitation (United Nations, 2024).
- Goal 7: Affordable and clean energy – ensure access to affordable and sustainable energy (United Nations, 2024).
- Goal 8: Decent work and economic growth – promote sustained, inclusive, and sustainable economic growth, full employment, and decent work (United Nations, 2024).
- Goal 9: Industry, innovation, and infrastructure - build resilient infrastructure and promote sustainable industrialisation (United Nations, 2024).
- Goal 10: Reduced inequalities – reduce inequality within and among countries.
- Goal 11: Sustainable cities and communities – make cities and human settlements inclusive, safe, resilient, and sustainable (United Nations, 2024).
- Goal 12: Responsible consumption and production – ensure sustainable consumption and production patterns (United Nations, 2024).
- Goal 13: Climate Action – take urgent action to combat climate change and its impacts (United Nations, 2024).
- Goal 14: Life below water – conserve and sustainably use the oceans, seas, and marine resources (United Nations, 2024).
- Goal 15: Life on land – protect, restore, and promote sustainable use of ecosystems and forests, and combat desertification. Halt and reverse land degradation and biodiversity loss (United Nations, 2024).
- Goal 16: Peace, justice, and strong institutions – promote peaceful and inclusive societies for sustainable development (United Nations, 2024).
- Goal 17: Partnerships for the goals – strengthen and revitalise the Global Partnership for Sustainable Development (United Nations, 2024).

2.2.3. The Construction Sector and Climate Change

The construction sector contributes to the global annual carbon emissions. The 2022 Global Status Report for Buildings and Construction found that the construction sector accounts for approximately 37% of energy-related carbon dioxide emissions (UNEP, 2022), which is more than a third of the global carbon dioxide emissions. Urbanisation is rising and the population is set to continue growing. Therefore, the demand for building materials to construct global cities is also rising (United Nations Environment Programme, 2023).

Many of the Sustainable Development Goals (SDGs), as described in the previous section, are linked directly to the construction sector. Whilst some of the Goals are indirectly linked to the sector, such as Goal 8 for decent work and economic growth, some have more direct links. This is further motivation and reason for the construction sector to innovate and strive for decarbonisation and improving the overall sustainability of the industry.

Goals 9, 11, 12, 13, and 15 are directly linked to the performance of the construction sector. Whilst there are other sectors which of course must contribute, the construction sector must also play its part. The construction sector must strive to innovate and build sustainable and resilient infrastructure to meet Goal 9. It must adopt sustainable and modern methods of construction in order to build sustainable infrastructure, cities and settlements, as per the targets of Goal 11. As many of the raw materials used in construction are derived from quarrying, the sector must ensure that these practices are carried out in a safe and environmentally friendly way, that do not harm ecosystems and habitats, as required for Goal 15.

Goals 12 and 13, which call for the responsible consumption and use of materials and for climate action respectively, relate specifically to the scope of this research project. Accurately designed reinforced concrete elements require a smaller volume of materials to perform their given tasks. This reduction in the quantity of required materials assists directly in meeting the targets of Goal 12. Subsequently, this reduces the embodied carbon dioxide emissions from the structure by eliminating the use of unnecessary materials, which is a target of Goal 13.

The life cycle carbon emissions of a building are the sum of the embodied carbon dioxide emissions from the construction stage and the operation carbon dioxide emissions from the operational stage. The embodied carbon dioxide emissions come from the extraction, manufacture, transport, and construction of building materials. The process carbon dioxide emissions are from the energy use within the building (United Nations Environment Programme, 2023).

Within the 37% share of emissions from the construction sector, 75% are from operational emissions and 25% are from embodied carbon dioxide emissions, as shown in Figure 2. Put simply, 6% are embodied carbon emissions from the most used building materials, namely concrete, steel, and aluminium Until recently, the focus was on reducing the operational carbon dioxide emissions of a building. However, efforts have lagged in minimising the embodied carbon dioxide emissions from building materials (United Nations Environment Programme, 2023). Therefore, to minimise the emissions from the construction sector, the embodied carbon of these building materials must be addressed. As shown in Figure 2, if the construction sector continues to operate in a ‘business-as-usual’ basis, the embodied carbon of building materials is projected to rise from 25% to nearly half (49%) of the life-cycle emissions of a building. The operational carbon is predicted to shrink due to the decarbonisation of electricity grids and as building operations become more efficient (United Nations Environment Programme, 2023).

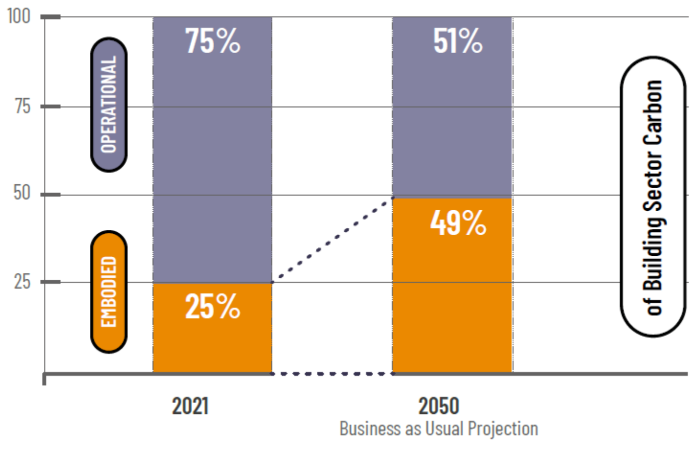

Figure 2: Projected contributions from operational and embodied carbon emissions within the construction sector (United Nations Environment Programme, 2023)

2.2.4. Building Lifecycles and Life Cycle Assessments

A Life Cycle Assessment (LCA) is a concept that has recently become popular in industry and in research that aims to quantify the environmental impacts of a product/material. Primarily, an LCA quantifies the carbon emissions of a product over its life span. An LCA considers all the inputs, the products, and the possible environmental impacts of a product or material over its full life cycle. It therefore includes all factors from the extraction of the raw materials until the final disposal of the product, including manufacture, distribution, and use (Think Wood, 2021). These results are reported in $kgCO_2$, which is the quantity of carbon dioxide emitted into the atmosphere from a product over its life cycle. Alternatively, the mass of carbon dioxide equivalents can be provided, denoted as kgCO_2-eq, which is a measure of the quantity of carbon dioxide plus other greenhouse gases emitted, such as methane, nitrous oxide, etc. (Sustainable Business Toolkit, 2023).

The LCA process can be applied to a whole building, and these studies are called Whole Building Life Cycle Assessments (WBLCAs). The different life cycle stages, called modules, are shown in Figure 3. These modules make up the embodied carbon and the operational carbon of a building's entire life cycle, as discussed in Section 2.2.3. For example, Module A encompasses the raw material extraction, transportation, and manufacture of a product or material such as cement, as well as the construction process, such as the mixture and casting of concrete for a concrete frame building. The operational carbon is also quantified in Modules B6 and B7.

As part of this research project, the Embodied Carbon Calculator created by Circular Ecology was used to determine the embodied carbon of the designed elements. The boundaries of the calculator are normally set as cradle to site. However, the transport distance was set as zero, which allowed for a cradle to gate investigation. A cradle to gate investigation includes Modules A1 to A3.

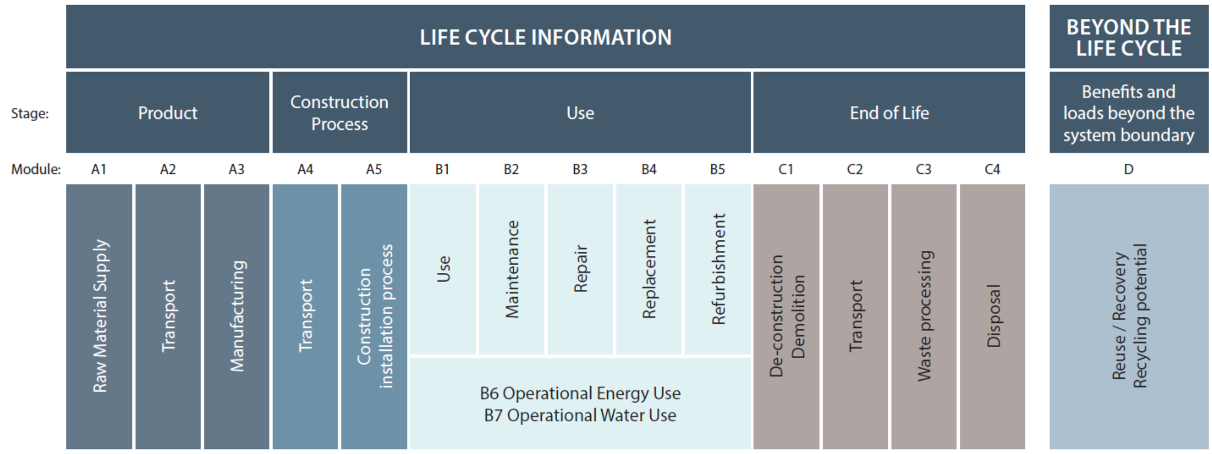

Figure 3: Life cycle stages used in a Whole Building Life Cycle Assessment (The Concrete Centre, 2021)

The embodied and operational carbon emissions from a building over the entirety of its life span are shown in Figure 4. From this graph, it is evident that reducing the embodied carbon dioxide emissions from the building materials used in the construction process will play a pivotal role in reducing the carbon dioxide emissions from the construction sector as a whole.

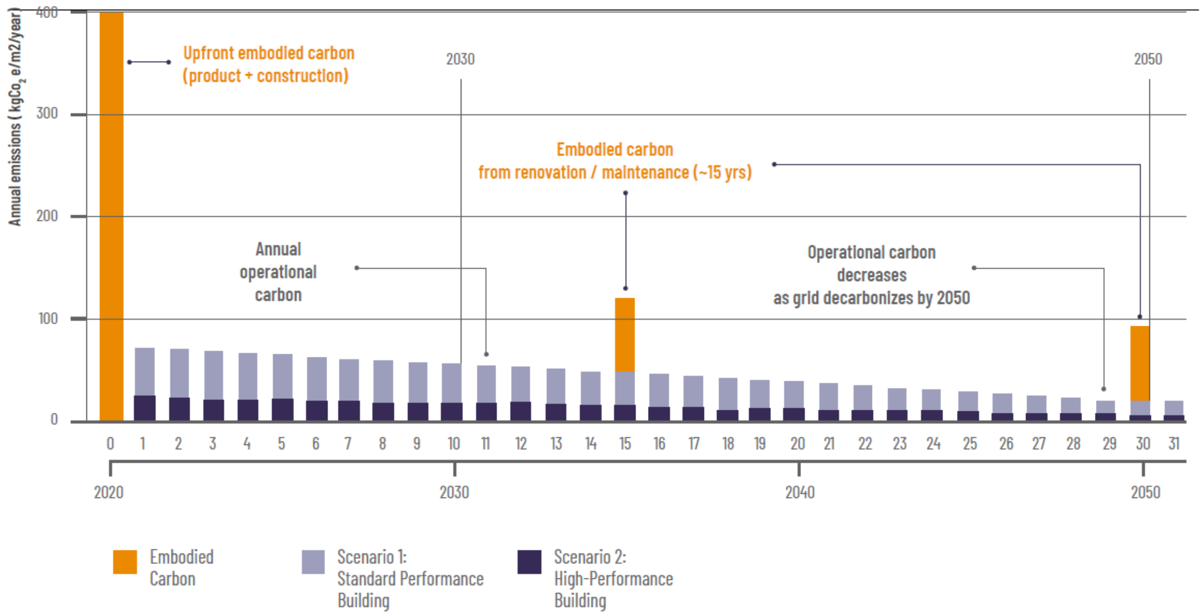

Figure 4: Embodied and operational carbon over a building's life cycle (United Nations Environment Programme, 2023)

LCA's also quantify environmental effects according to six main categories, known as impact indicators, as listed below. For this research, only the Global Warming Potential (GWP) indicator was considered.

- **Global Warming Potential GWP** (kgCO_2-eq) – quantifies the extent to which a product contributes toward climate change. This depends on the amount of carbon dioxide and other greenhouse gases associated with the products manufactured (EPD Ireland, 2018).
- **Ozone Depletion Potential ODP** (kgCFC11-eq) – quantifies the damage to the ozone layer by the emissions of ozone-depleting gases, such as chlorofluorocarbons, etc. (EPD Ireland, 2018).
- **Acidification Potential AP** (kgSO2-eq) – measures the extent to which the product acidifies the environment (which results in acid rain). Sulphur dioxide is an example of an acidic gas (EPD Ireland, 2018).
- **Eutrophication Potential EP** (kgPO4-eq) – this is a measure of the phosphate and nitrate concentrations in water which can lead to eutrophication and algae bloom. This is harmful to aquatic life (EPD Ireland, 2018).
- **Photochemical Ozone Creation Potential (POCP)** – this is the measure of smog-causing gases, such as nitrogen oxide (EPD Ireland, 2018).
- **Abiotic depletion** – this is an indicator of the availability of non-renewable resources, which is in decline (EPD Ireland, 2018).

2.3. Reinforced Concrete as a Building Material

Concrete is a key material used in the construction sector and is one of the most widely used engineered materials in the world, and the second most-used substance on Earth, after water (Scrivener, John, & Gartner, 2017). Concrete, which performs well in compression but not in tension, is generally cast on site and has steel reinforcing bars embedded therein, to ensure that the member does not fail in tension. This is referred to as reinforced concrete.

Cement is the primary component of concrete. Cement is combined with water and aggregates to produce concrete. Due to the versatility, workability, and durability of concrete, it is the most popular and practical choice for construction projects worldwide. Concrete was one of the primary construction materials used during the rise of urbanisation and remains a widely used construction material today. The ubiquitous use of cement also derives from the fact that its important components, calcium oxide, commonly known as lime, (CaO) and silica (SiO_2), are the most abundant mineral compounds on the surface of the earth. They are also cheap to mine and manufacture.

Unfortunately, cement used in concrete has a high environmental footprint and accounts for a significant portion of the carbon emissions on Earth. The production of concrete releases carbon dioxide and other greenhouse gases into the atmosphere; it uses a substantial amount of water in the production process; it relies heavily on transportation which uses fossil fuels; and it promotes the practice of quarrying, which is harmful to local ecosystems and the environment. To extract and prepare the aggregates used in concrete, which make up approximately 80% of the overall mass of concrete, the aggregates must be quarried, crushed, and transported to manufacturing facilities, and to construction sites.

Cement and concrete industries are therefore actively researching and developing technologies that will both protect the environment and meet the infrastructure needs of a continually changing, urbanised world (Naik & Mariconi, 2005). According to the Net Zero Carbon Guide, the most effective way to reduce embodied carbon in the construction sector is to reduce the quantity of the materials required, specifically if they have high embodied carbon, such as concrete (Pochee, n.d.). A reduction in the concrete required for an element to perform its intended function can lead to a reduction in the embodied carbon of the element, provided that a large volume of reinforcement is not required to sustain the loads.

2.3.1. Significance of Concrete as a Construction Material

Concrete is a key material used in the construction sector and accounts for the construction of everything from single-home dwelling units to large apartment complexes, from dam walls to skyscrapers. This is due to its significant strength, durability, and workability. Effective concrete design is therefore essential to ensure that the necessary properties required by the structures are achieved. Concrete is a universally used material as it is used by laypersons and highly skilled persons alike (Scrivener, John, & Gartner, 2017). This is due to its affordability, availability, and its relative ease of use.

While the carbon dioxide associated with the production of concrete has come under a lot of scrutiny in past years, the benefits of concrete should not be forgotten. The most notable benefit is that concrete is known to last for centuries (IEA , 2009). This makes concrete structures the most appealing from a cost perspective. From a sustainability perspective, there is the added benefit that the emissions can be balanced over the full life cycle of the structure. In other words, even though the embodied carbon of structure is high during the construction phases, these can be spread over the design life of the structure if replacement is not necessary.

Other construction materials such as steel and timber are also available for structural designs. Steel remains a popular construction material across the world due to its significant strength, especially in tension, where concrete is severely lacking. Reinforcement has a characteristic tensile yield strength of 500 MPa, whereas concrete has a mean tensile strength of 2.9 MPa for a C30/37 concrete mixture (Eurocode Applied, n.d.). Other factors may be more important to the design of a specific structure, such as weight, buildability, etc. An additional advantage of steel over concrete is that the elements can be manufactured off-site (ESC Group, 2020).

There has been a rise in the popularity and experimentation of mass timber buildings. Mass timber, such as the commonly used engineered wood called cross-laminated timber (CLT), can be used as a structural load-bearing member (Structurlam, 2017). However, while steel, mass timber, and other construction materials have several benefits, they cannot replicate the versatility of concrete, and the ability for a structural element to be cast on site. This emphasises the value of reducing the embodied carbon and other negative environmental impacts of concrete for its continued future use as a sustainable construction material.

2.3.2. The Background to Concrete as a Construction Material

The three primary constituents of concrete are Portland cement (PC), coarse and fine aggregate, and water. The production of PC releases a large amount of carbon dioxide. For 1 tonne of Portland cement clinker, approximately 1 tonne of carbon dioxide emissions and other greenhouse gases are also released (Naik & Mariconi, 2005). The other pollutants which are released are particulate matter, nitrogen oxide, and sulphur dioxide (ACMP, 2011). The use of concrete in the construction industry accounts for approximately 5% of the total greenhouse gas emissions released each year (Scrivener, John, & Gartner, 2017).

Additionally, when extracting limestone, also known as calcium carbonate, from a quarry, degradation of the environment occurs. Limestone is an ingredient used to produce PC clinker. The course and fine aggregate required in a concrete mix are also obtained through quarrying practices. Quarrying and open cast mining can lead to erosion, loss of usable land, damaged landscapes, and deteriorated water quality from pollutants and sediment released into surface water and groundwater. Quarries also produce large amounts of noise and dust pollution which can influence human health (British Geological Survey, n.d.).

Water is one of the primary constituents of concrete. It reacts chemically with the cement in the hydration process and promotes the strength gain of the concrete. Water quantities in the mix are controlled by the water/cementitious material ratio (w/c ratio) to achieve specified characteristics.

2.3.3. Manufacturing Cement

Cement is the 'binding' component that is used in concrete production. Two aspects of cement production are responsible for the high concentrations of carbon dioxide released. The first is the carbon dioxide released in the chemical decomposition of lime by heat, and the second is the energy needed to produce the cement clinker.

The primary raw material of Portland Cement (PC) clinker is limestone, which is mined in a quarry usually situated near the cement plant. Additional materials, such as shale or sand, are included in production to provide more alumina, gypsum, and silica which improve the chemical composition of the final product. These raw materials are then crushed for ease of transport and processing into pieces of approximately 10 cm (IEA , 2009). The process for manufacturing cement is illustrated in Figure 5.

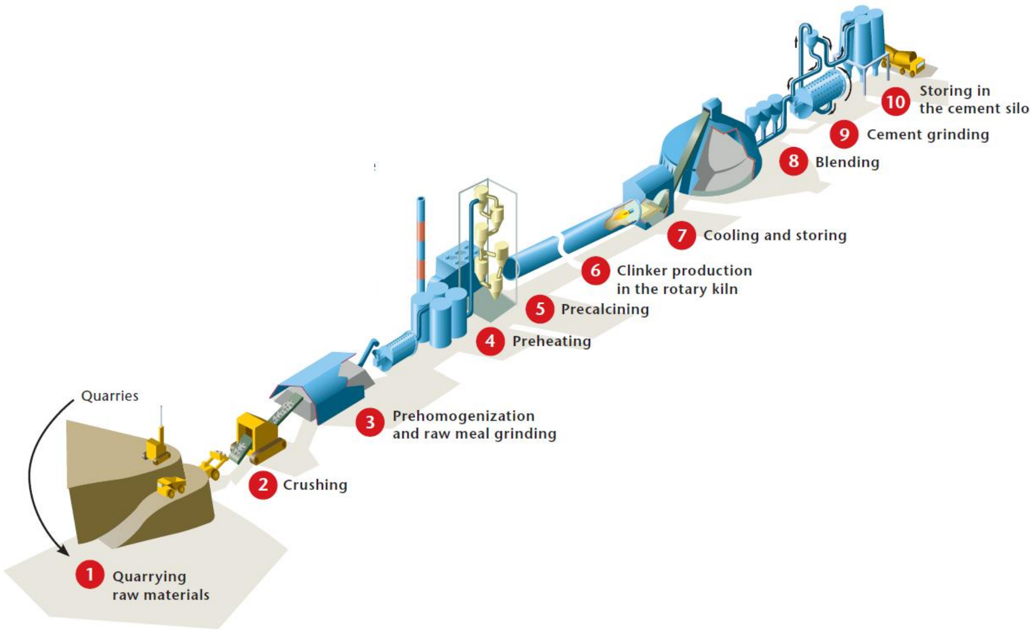

Figure 5: The cement manufacture process (IEA , 2009)

The raw materials are then mixed and proportioned to obtain an optimal mixture based on chemical composition. This process is called pre-homogenization. After this, the raw mixture is milled together. This process is strictly monitored and controlled (IEA , 2009).

The raw mixture is preheated as it passes through several vertical cyclones where the particles make contact with the exhaust gases from the kiln, which preheats the mixture. This process ensures that the chemical reactions occur quickly and completely. The moisture content of the raw meal affects the number of kiln cyclones required in the process. The kiln can have up to six cyclones in series to maximise heat recovery with each stage (IEA , 2009). The next step in the process is pre-calcining. Calcination occurs when limestone decomposes into lime, and this takes place in the pre-calciner. There is a combustion chamber positioned in the middle of the pre-heater and kiln. It is at this stage where the limestone ($CaCO_3$) chemically decomposes into lime (CaO) and this combination of limestone and heat emits 60-65% of the total emissions in the cement manufacturing process (IEA , 2009). One tonne of PC clinker produces approximately 1 tonne of carbon dioxide emissions and other greenhouse gases (Naik & Mariconi, 2005).

After pre-calcination, the mixture enters the kiln, which can reach temperatures of 1450ºC. Known as rotary kilns, they rotate at approximately 3 to 5 revolutions per minute, which allows the material to tumble downwards, where it gets progressively hotter. This heat prompts physical and chemical reactions that partially melt the mixture. It then falls into a grate cooler, where it is cooled and forms the hard, granular material known as clinker (IEA , 2009).

It is at this point that the clinker can be mixed with other mineral components. All cements are blended with 4-5% of gypsum. Gypsum is used in cement to control the setting time (or rate of hardening), which is an important aspect of concrete mixtures, especially for concrete cast on-site (IEA , 2009).

Supplementary Cementitious Materials (SCMs) are included in the mix in this stage, to form "blended cement" (IEA , 2009). SCMs are usually chosen based on the desired concrete characteristics, or to save on the costs of production by reducing the amount of cement clinker used. This is an effective way to reduce the carbon dioxide released in the production of concrete by limiting the amount of Portland cement (PC) used in the cement mixture and partially replacing it with SCMs, which are usually waste materials from other industries. This both reduces the carbon dioxide emissions and reduces wastage. Some examples of SCMs are ground granulated blast furnace slag, pulverised fly ash, silica fume, and calcined clay. This technique has been used extensively over the past few decades not only because of sustainability, but also because of the positive effects that SCMs have on the performance, durability, and production costs of concrete (Samad & Shah, 2017).

The cooled clinker and gypsum mixture, along with any SCMs, is then ground into a grey powder, which is stored in cement silos (IEA , 2009).

2.3.4. Common Cement Types Used in Concrete Mixtures

Several cement types can be specified for use on-site, and concrete designers need to be aware of the available types, and the differences between each. The cement types have been classified according to the *British Standard EN 197-2 Cement Assessment and Verification of Constancy of Performance* (European Standards, 2020). The naming method of the cement is used to designate what proportion of the cement was replaced with SCMs. Table 1 shows the naming convention and the typical replacement range.

Table 1: Common cement types used in industry and their naming conventions (VV/M Cement, 2013)

Cement Name		Description
CEM I	Portland cement	This is the cement which is made up entirely of PC (with a maximum of 5% of other materials)
CEM II	Portland hybrid cement	This cement has a minimum of 65% PC, and up to 35% of the blend is fly ash, slag, etc.
CEM III	Blast furnace cement	A higher replacement of slag is blended with the PC, in the range of 40-90% replacement.
CEM IV	Pozzolanic cement	This is a blended cement of PC and pozzolans.
CEM V	Composite cement	This comprises a higher replacement percentage with slag or pozzolanas.

2.3.5. Hydration of Cement to Make Concrete

Concrete is formed when Portland cement (PC) and Supplementary Cementitious Materials (SCMs) are mixed with aggregates and water. The reaction between cement and water is called hydration and creates a hardened cement paste. PC is comprised of tricalcium silicate (C_3S) and dicalcium silicate (C_2S) which form calcium-silicate-hydrate (*C-S-H*) and calcium hydroxide ($Ca(OH)_2$) when they react with water. $Ca(OH)_2$ is commonly known as lime. This reaction is shown in Equation 1. C-S-H is amorphous and responsible for the cementitious properties, and therefore the strength characteristics. The greater the number of C-S-H particles in a mixture, the higher the strength and durability characteristics. $Ca(OH)_2$ is a crystalline substance that does not contribute to the strength of concrete (Barron & Johnson, 2012).

$$Cement+Water=Calcium\ Silicate\ Hydrate+Calcium\ Hydroxide \qquad (Equation\ 1)$$

2.3.6. Steel Reinforcing Bars

Reinforcing bars are used in reinforced concrete designs to increase the combined tensile strength of the element or structure. They assist the concrete in withstanding the tensile, bending, torsion, and shearing loads of the structure. Reinforcing bars, known as rebar in industry, are almost always made from steel, as steel has similar thermal expansion properties to concrete which results in fewer issues with temperature changes (Metal Supermarkets, 2021).

Steel is produced from one of two standard processes. One is the Blast Furnace - Basic Oxygen Furnace (BF-BOF) route, which is often referred to as the ‘primary’ path because it is used to create new ‘virgin’ steel. The other is the Electric Arc Furnace (EAF) route, which is referred to as the ‘secondary’ path because it is often used to recycle steel scrap (Eurofer, 2020).

Blast furnaces convert iron-ore into iron by a chemical process called reduction. Iron-ore contains both iron and oxygen. Carbon is used as a reducing agent to separate the iron from the oxygen. It is during this process that the oxygen combines with the carbon to form carbon dioxide. After the iron has been extracted, the Basic Oxygen Converter turns the iron into steel. In this step, oxygen is blown into the converter holding the liquid iron. This burns away the unwanted elements and turns the iron into steel. As per the European Steel Association, the large blast furnaces in Europe can create up to four million tonnes of iron per annum, and approximately 60% of the steel produced in Europe comes from this process (Eurofer, 2020).

Electric Arc Furnaces differ from blast furnaces as their primary input material is scrap steel collected for recycling. Other than scrap steel, EAFs can smelt solidified iron or sponge iron (Eurofer, 2020). An EAF has a capacity of 1.5 million tonnes per annum, and approximately 40% of the steel produced in Europe is derived from this production route.

2.3.7. Types of Reinforcing Bars

There are several types of reinforcing bars that can be used in reinforced concrete designs. These are broken down into mild steel bars and high-yield steel bars. The main difference between the two is that high-yield steel has a significantly higher strength than mild steel. This makes high-yield steel a more popular choice for reinforcing bars as it has a higher tensile strength. Mild steel reinforcing bars differ in composition from high-yield bars as they contain less carbon by weight (Madhu, 2019).

High-yield steel bars are further broken down into hot-rolled steel bars and cold-worked steel bars. Figure 6 is from EN1992 and shows the stress versus strain curves for hot-rolled steel and cold-worked steel. Hot rolled steel is produced at temperatures above 530°C whereas cold worked steel is processed at average temperatures. Hot-rolled steel is not as strong as cold-worked steel, and since it cools after processing, the final shape will likely have variations. Therefore, cold-worked steel can be formed more precisely. Hot-rolled steel is therefore usually unsuitable for precise applications but is the steel type of choice for general construction practices (Heaton Manufacturing, 2020).

High yield reinforcement bars have gained popularity and are predominantly used in RC construction today. High yield steel reinforcement bars have a higher upfront cost compared to mild steel reinforcement but provide cost savings in the long term. This is due to their higher strength and improved durability which result in a reduced quantity of bars required to sustain the load (Star Steels Limited, 2023).

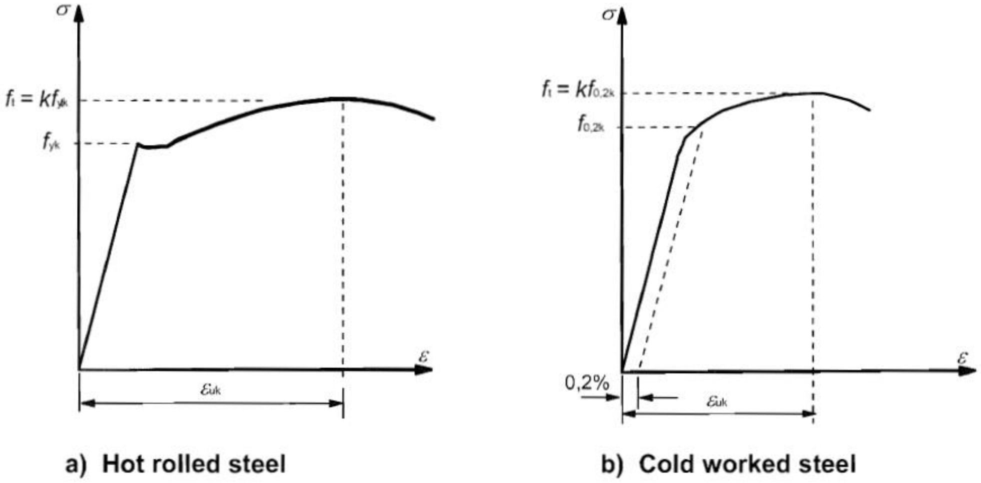

Figure 6: Stress vs strain diagrams of typical reinforcing steel (hot rolled and cold worked steel) (The European Union, 2004)

2.4. Mitigating Embodied Carbon in the Construction Sector

The embodied carbon of reinforced concrete must be reduced to ensure that concrete remains the construction material of choice. P.K. Mehta noted that a reduction in embodied carbon from the construction industry will require a restructure within the concrete industry from a culture of acceleration to a culture of resource productivity and optimisation. The key measures for reducing the environmental impacts of concrete are increasing the durability of the concrete and conserving high embodied carbon materials as well as energy for concrete production (Mehta, 2001).

To reduce concrete's embodied carbon, cement is often blended with SCMs. Reducing the clinker-to-cement ratio by using SCMs can decarbonise cement, as SCMs have a significantly lower embodied carbon dioxide emissions than Portland cement, as shown for comparison in Table 2 (United Nations Environment Programme, 2023). However, the effects of SCMs are not included into the scope of this research project so as not to skew the results.

Table 2: Comparative embodied carbon of various supplementary cementitious materials (SCMs) (Samad & Shah, 2017)

Material	Embodied CO_2 (kg/tonne)
Portland cement (CEM1)	913
Limestone	75
Ground granulated blast furnace slag (GGBS)	67
Pulverised fly ash (PFA)	4

There are several other options available to the construction sector for reducing the embodied carbon dioxide emissions from the construction phase. These include, but are not limited to:

- Reusing and retrofitting existing buildings to avoid the need for new concrete, thus reducing the embodied carbon of the project.

- Using precast members with optimised dimensions which are cast in a factory to improve quality control and reduce wastage from the construction sector (Bachmann, 2017).
- Using prestressed reinforced concrete members. Prestressed concrete members are stronger in tension than regular reinforced concrete members and can be used to produce floors or beams with longer spans. This reduces the volume of concrete required in a structure, which in turn is a way of decarbonising the structure (Waly, et al., 2018).
- Using bio-based materials, such as timber, wood, and bamboo, which are truly renewable may represent the best and most radical way to decarbonise the construction sector (United Nations Environment Programme, 2023).
- Optimising the dimensions of structural members.

For the purposes of this research, only the effects of optimising the dimensions of structural elements were investigated. This study was used to determine the carbon dioxide savings purely from the optimization of the element dimensions and the reductions in material quantities, without considering the quality and composition of those materials. Therefore, SCMs were introduced into the scope of this research.

The way that a structure is designed and constructed can have significant environmental impacts, due to the quantities of materials required, the quality and composition of the materials used, the durability of the structure, the intended service life of the structure, etc. It is therefore important to consider the below factors in the structural design of a given element in order to ensure that the structure is designed to minimise embodied carbon emissions.

- **Design loads:** The structure must meet the minimum design load requirements to be deemed safe and sustainable. This includes permanent, imposed, service, dynamic, and wind loads. Accurate representation of the design loads leads to a less drastically over-specified structure, which in turn relates to a lower concrete mass used on a project. The continual research of safety factors and other constants must be prioritised to get a better understanding of structures and to minimise over-specification of concrete elements. This improves the overall embodied carbon of the structure as less concrete is consumed by the site (Lemay, Lobo, & Obla, 2013).
- **Durability:** If a structure is more durable, it requires less maintenance which results in fewer repairs. Durability depends on effective designs and concrete mix designs (Lemay, Lobo, & Obla, 2013).
- **Constructability:** If care is taken in this phase, there will be less wastage of building materials. For example, if an unrealistic slump is proposed, this may affect pumpability and result in the wastage of the prepared concrete (Lemay, Lobo, & Obla, 2013).

Due to atmospheric pollution, poorly built structures deteriorate faster. Concrete of poor quality can exhibit many cracks and honeycombing, which can require expensive repairs Structures that fail because of adverse weather usually do so due to pre-existing cracks (Mehta, 2002). Increasing the durability of concrete structures is a solution for improving resource production in the concrete industry.

This is achieved through efficient concrete design and material usage. Efficient concrete design limits the volume of concrete and reinforcing steel required for the structure to perform as intended. The use of reinforcing steel in reinforced concrete comes at a high cost to the project as it is expensive, and if it is not properly laid and protected, it can have negative effects on the durability of the structure due to rusting. This effect on the durability of the structure can greatly impact the service life of the structure. It is also costly to the environment as greenhouse gases are released during the production of steel. If the quantity of reinforcing steel is significantly reduced, then the cost of the structure and the embodied carbon of the structure would both be greatly reduced.

P.K. Mehta emphasises the importance that the durability of a structure has on its service life. Improved durability comes from decreasing the porosity and permeability of the concrete. This in turn lessens the corrosion of the reinforcement used in the structure by limiting water ingress. It is also important to consider and improve crack resistance in order to ensure the structure reaches its intended service life. Improved concrete durability also means less frequent repair of infrastructure and hence, reduced carbon dioxide output.

2.4.1. Embodied Carbon of Reinforced Concrete

Concrete

As discussed in Section 2.3.4, there are different types of cements available on the market. These depend on the proportion of Portland cement in the mixture. For the purposes of this research, CEM I cement was used in the designs in order to eliminate the effects of SCMs on the embodied carbon results.

CEM 1 is the standard Portland cement which is made entirely of PC and has a maximum of 5% of other materials blended therein. CEM I has an embodied carbon of 913 kgCO_2e/tonne. By mass, concrete has a relatively low embodied carbon compared to steel (ARUP, 2023).

Steel Reinforcement

Reinforcing steel is used in reinforced concrete to resist tensile forces. This is required because whereas concrete has high compressive strength, it has poor tensile strength. However, the use of reinforcing steel in reinforced concrete comes at a high cost to the project as it is expensive, and if it is not properly laid and protected, it can have negative effects on the durability of the structure due to rusting. This effect on the durability of the structure can greatly impact the service life of the structure.

It is also costly to the environment as greenhouse gases are released during the production of steel. Steel production impacts the environment negatively in several different ways. This includes emitting greenhouse gases into the atmosphere, producing wastewater contaminants, and producing hazardous and solid wastes (GreenSpec, n.d.).

If the quantity of reinforcing steel is significantly reduced, then the cost of the structure and the embodied carbon of the structure would both be greatly reduced

As per the Net Zero Carbon Guide, steel from blast furnaces has an equivalent embodied carbon of 2800 kgCO_2e/tonne, whereas steel from an Electric Arc Furnace is around four times less than this, at 800 kgCO_2e/tonne. This is because electric arc furnaces use a higher percentage of recycled steel in the production of new steel. Therefore, less virgin materials are required. An additional benefit to the usage of Electric Arc Furnaces is that they can be powered by low-carbon electricity (Pochee, n.d.).

However, there is only a finite amount of scrap steel available, and projects or countries that use up more of the recyclable steel leave less available to other projects and countries. It is therefore suggested that designers take a global view when regarding the sustainability of designs and use an average of 40% scrap steel content (Pochee, n.d.).

2.4.2. Embodied Carbon Calculator for Concrete

ICE is a materials database of the embodied carbon of different products and materials used within the construction sector. As part of ICE, the *Embodied Carbon Calculator for Concrete* software was developed (Circular Ecology, 2019). This software was used to calculate the embodies carbon of the reinforced concrete elements designed in this research project. This software takes multiple factors into account, such as the volume of concrete and reinforcement in the element, as well as the concrete mix design.

As part of this research project, the Embodied Carbon Calculator created by Circular Ecology was used to determine the embodied carbon of the designed elements. The boundaries of the calculator are normally set as cradle to site. However, the transport distance was set as zero, which allowed for a cradle to gate investigation. This corresponds to the LCA Modules A1 to A3, as seen in Figure 3.

The Inventory for Carbon and Energy (ICE) materials database provides a value of 1.4 kgCO_2e per kilogram of steel produced. However, this value is derived using the European average recycled content of 59% (Hammond & Jones, 2011).

2.5. Evolution of Structural Design Methodologies

A structural designer must ensure that a structure is fit for its intended purpose, that it is safe, and that it is economical and durable. However, this is complicated due to uncertainties in loading, material strengths, and structural behaviour, which is specifically true for reinforced concrete elements, as the properties of the concrete components must be taken into account as well as the steel reinforcing bars. Over the centuries, there have been leaps in the understanding of structural material properties, loading, and structural failure mechanisms. However, it is still impossible to guarantee the absolute safety of a structure. It is therefore important for risks to be mitigated and ensure that the risk of failure is small (Kumar & Kumar, n.d.).

Structural design methods have been used to create a structure that can resist applied loads without failing, as well as resist excessive deformations over its service life. Structural design methods are built into building codes. Building codes, or structural design codes, are a set of guidelines that ensure that the certain important standards are met in the construction industry. Building codes are in place to protect the public, but they are also the primary influence on the environmental sustainability of cities, and how buildings are designed, built, and maintained (Anwar & Najam, Progression of Structural Design Approaches: Working Stress Design to Consequence-Based Engineering, 2015). The design of an element is dependent on the building codes and the forces applied to the element. These forces dictate the design loads of the element.

Amongst others, the loads that are applied to a structure are gravity, wind loads, service loads, etc. Loads lead to actions in structural elements, such as shear forces and bending moments. These actions lead to deformations in the element, such as rotation, shortening, etc. Deformations lead to strains, which are the deformations of the element at its cross-section. Strains can induce stresses in the material (Anwar & Najam, Progression of Structural Design Approaches: Working Stress Design to Consequence-Based Engineering, 2015). Therefore, stress-strain models of a particular material are an important design tool. Over time, there have been three main design methodologies used in building design codes.

The oldest reinforced concrete design method is known as the Allowable Stress Design (ASD). This method assumes that the structure acts in a linear elastic manner. Restricting the stresses in the material ensured the safety of the structure. This was done by keeping the expected working loads of the structure well below the permissible stresses. Permissible stresses are significantly lower than the ultimate strength of the material, to ensure safety. For example, for a concrete cube compressive strength of 30 MPa, the permissible strength in the concrete is only 7.6 MPa. This is justifiable due to the assumed linear elastic behaviour of the material. A safety factor is defined as the ratio of the material strength to the permissible stress (Anwar & Najam, Progression of Structural Design Approaches: Working Stress Design to Consequence-Based Engineering, 2015).

Over time, the understanding of the behaviour of reinforced concrete at ultimate loads improved. Thus emerged the Ultimate Strength Design method, and this was used as an alternative to the ASD method. This method arose due to the evolution of material theories and the consideration of the nonlinear stress-strain curves of both concrete and steel. In this method, an appropriate load factor was introduced. The load factor is defined as the ratio of the ultimate load to the working load. It can fall in the range of 1.2 to 2. Different load factors can be assigned to different loads under a combination of loading actions. This leads to more smaller cross-sectional areas of beams and columns, especially if high-yield reinforcing steel is used. The setback of this method is that the good strength performance at the ultimate loading does not translate into satisfactory serviceability performance of the element or structure. The serviceability performance is based on service loads, and not the ultimate load. Therefore, the elements designed using this method deflected and cracked excessively due to the high strength and slender sections (Anwar & Najam, Progression of Structural Design Approaches: Working Stress Design to Consequence-Based Engineering, 2015).

The latest design methodology used is the Limit State Design method. The ASD method was based on calculations at the serviceability load conditions only and the Ultimate Strength Design method was based on ultimate load conditions only. The Limit State Design method takes both conditions into account and ensures safety at the ultimate load but serviceability at the working loads (Anwar & Najam, Progression of Structural Design Approaches: Working Stress Design to Consequence-Based Engineering, 2015). A “limit state” is the state of imminent failure after which point the structure cannot perform as intended, either in terms of safety or in terms of serviceability. Therefore, there is more than one safety factor in this methodology: one for the ultimate loads and one for the serviceability loads. These are related to two limit states, namely the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS). ULS deals with aspects such as strength, overturning, buckling, etc. SLS deals with the discomfort to occupants or malfunction that arises from deflection, cracks, loss of durability, etc. (Anwar & Najam, Progression of Structural Design Approaches: Working Stress Design to Consequence-Based Engineering, 2015)

2.6. Reinforced Concrete Design Codes

Throughout history, professional engineering bodies have led the way forward in the development of guidelines and codes, along with national legal authorities, academic studies, and practicing engineers. Structural design codes inform a practicing engineer what limits should be applied to each structural member. It is therefore important that structural design codes are kept updated with the latest developments to allow for progress and alternative strategies/approaches (Taylor & Burgoyne, 2008). The structural design codes provide a system for regulation within the construction sector. Essentially, they put the knowledge of material properties to safe structural use by applying the relevant structural theory. The codes are meant to strike a balance between safety and economy. However, it should be noted that safety should not be compromised (Taylor & Burgoyne, 2008).

The Institute of Concrete Technology (ICT), a professional engineering institute, was founded in 1908 in London, and it comprised engineers, architects, and other individuals. One of the primary goals of the ICT was to prepare and implement a set of rules that could be used to govern the use of reinforced concrete, which was a new material at the time. Before these rules were put in place, the use of reinforced concrete was mostly managed by contractors with material expertise, but there was no robust system of codes aimed at setting a design precedent (Taylor & Burgoyne, 2008). At the time that the ICT was founded, building regulations had existed for many years. However, this was the starting point for standardising the use of reinforced concrete in the European Union (Taylor & Burgoyne, 2008).

In 1908, Hooke, Young, and Euler's elastic theory started being used in structural design, which states that there is a proportional relationship between stress and strain in a linear elastic material. However, there were doubts as to whether elastic theory was suitable for reinforced concrete. Due to this, safety factors were specified as being quite high. It is for this reason that buildings constructed during that period are still functional today, and yet are considered over-designed. At the time, design stresses were low, and conditions such as deflection and vibration were hardly considered. Therefore, the large safety factors compensated for the lack of knowledge during the period (Taylor & Burgoyne, 2008).

In the 1930s, an important change to structural theory occurred. This was due to the ability to measure stresses in real structures by using strain gauges, which led to the observation that steel-framed structures did not act as predicted by the linear elastic theory. This led to the formalisation and utilisation of plastic theory by Hodge, Ducker, Prager, and Hill. Modern codes use plastic methods to limit the ultimate load capacity (UL) of the structure, but they use elastic methods to handle the stresses and deformations at design load (Taylor & Burgoyne, 2008).

Current design methods in use have been refined and safety factors reduced. This leads to greater economic benefits and improved sustainability due to the leaner structures which require fewer materials to meet the design standards. Therefore, accurate designs and correct material theories are essential as there is less margin for error (Taylor & Burgoyne, 2008).

2.6.1. Establishing the Eurocodes

Codes such as CP114 and CP115, which refer to reinforced concrete and prestressed concrete respectively, still relied on elastic theory and therefore had higher factors of safety. In the 1950s, the Federation International de la Preconstraint (FIP) and the Comite Europeen du Beton (CEB) created the first set of codes for structural concrete design, which were published in 1964, and a second edition was published in 1970. The CEB code introduced the Limit State Principles to engineers. This included the Serviceability Limit State, which states that the structure should not deflect significantly at the design load, and the Ultimate Limit State, which states that a structure should not collapse until a higher load is reached (Taylor & Burgoyne, 2008).

These limit state codes, such as the Code of Practice CP110 "The Structural Use of Concrete", allow for more flexibility of the variable loads and variable material properties. For example, concrete is more variable than steel and highway loads are more variable than hydrostatic pressures. These codes also allow for different consequences of failure, which is important in high-risk scenarios. Therefore, higher safety factors may be used in a nuclear power station than in a less risky project, such as a residential building. This prevents all projects from having to use the safety factors of the worst-case scenario (Taylor & Burgoyne, 2008).

The development of the Eurocodes began in 1975, and the first set of codes was published by the European Union (EU) Commission in 1984 with the help of a steering committee (Joint Research Centre, n.d.). National standards bodies work through the CEN (the European Standards Body), and the experts who are selected to produce the Eurocodes are chosen based on their expertise, but do not work under their national flag. The CEN specifies the rules regarding the timing, writing, review, translation, etc. of the Eurocodes. This has resulted in 10 Eurocodes being written, which comprise 58 parts. National Annexes are prepared by each country to specify the safety factors that should be used for their country (Taylor & Burgoyne, 2008).

The structural Eurocodes, which came into effect in 2010, are now recognised as the foundation for coordinated structural design in Europe. This has allowed engineers to tender for projects and to work on these projects across the continent (Murphy, 2020). The Eurocodes were drafted by technical teams with unique and different backgrounds. The Eurocodes are used to ensure public safety throughout the European Union; however, they are also a basis for international use (Taylor & Burgoyne, 2008).

The development of the second set of Eurocodes is currently underway by the European Commission. The revision of these codes is being overseen by the European Committee for Standardisation (CEN) technical committee (CEN/TC 250 – Structural Eurocodes) and the number of Eurocode parts is expected to rise from 58 to 64 to incorporate standards for retrofitting structures, the use of structural glass, and the structural actions required for atmospheric icing and waves and currents (Murphy, 2020).

2.6.2. National Annexes of the Eurocodes

Certain national aspects regarding safety factors are considered through the creation and use of National Annexes. This means that safety levels for buildings are controlled at a national level. This includes taking climatic conditions and procedures into account. These are addressed by Nationally Determined Parameters (NDPs) which allow countries to set their parameters and requirements. However, the Eurocodes also refer to "recommended values" which can be used in the absence of a National Annex.

The Eurocodes have been used in Ireland since March 2012 as the Irish Standard (I.S.) EN 1990 and I.S. EN 1999. The NDPs in Ireland are maintained and revised by the National Standards Authority of Ireland (NSAI) (Murphy, 2020).

2.6.3. Structural Eurocode Designations

The titles of the ten Structural Eurocodes are listed below. Their British Standards designations have been included next to each code.

- Eurocode 0: Basis of structural design (EN 1990)
- Eurocode 1: Actions on structures (EN 1991)
- Eurocode 2: Design of concrete structures (EN 1992)
- Eurocode 3: Design of steel structures (EN 1993)
- Eurocode 4: Design of composite steel and concrete structures (EN 1994)
- Eurocode 5: Design of timber structures (EN 1995)
- Eurocode 6: Design of masonry structures (EN 1996)
- Eurocode 7: Geotechnical design (EN 1997)
- Eurocode 8: Design of structures for earthquake resistance (EN 1998)
- Eurocode 9: Design of aluminium structures (EN 1999)

2.6.4. Historic European Structural Design Codes

To make an accurate comparison of the historic versus the current design codes, it is important to correctly establish which codes have historically been in use for the geographical region. A representation of the timeline of structural codes has been shown in Table 3. The year of publication for each code has been listed in the table. It should be noted that only the codes specifically relating to the construction of buildings have been included in the table, and those about the design of bridges, maritime structures, etc, have not been included, as they fall outside the scope of this research. As mentioned in Section 2.6.1, the development of the Eurocodes began in 1975, but the first set of Eurocodes only came into effect in 2010.

Table 3: Historic design codes (The Concrete Society, 2020)

Number	Design Code Name	Year Published
CP 114	The structural use of normal reinforced concrete in buildings	1948 (revised in 1957, published in metric units January 1969)
CP 115	The structural use of prestressed concrete in buildings	1959
CP 116	The structural use of precast concrete	1965
CP 110	Code of practice for the structural use of concrete	1972
BS 8110	Structural use of concrete, Part 1: Code of Practice for design and construction.	1985
EN1990 – EN 1999	Structural Eurocodes	2010

Before the British Standards (B.S.) were introduced, the Code of Practice (CP) standards were in use. B.S.8110, as shown in Table 3, was originally published in 1985 but has since been withdrawn. B.S.8110 is based on *Limit State Principles*, which is also used in the Eurocodes. However, its predecessor, CP114, was based on the older concept of *allowable stress design*. From Table 3, it is evident that CP114 was in use for a period of 24 years (1948 – 1972), whereas CP110 was only in use for 13 years (1972 – 1985). Therefore, it was decided for this research that the CP114 code would be analysed, as it was in practice for a longer period, and therefore, the number of structures designed using its methodologies would have been greater.

CP114 was originally published in 1948, revised in 1957, and re-published in metric form in 1969. It has, of course, since been withdrawn. It was originally withdrawn in 1972, however, its successor, CP110 was considered a cumbersome design tool, and therefore the CP114 code remained in practice for many years afterward. However, with the development of B.S.8110 in 1985, this code was finally surpassed (Beal & Thomason, 1986).

The current Eurocode and the withdrawn B.S.8110 standard are based on the Limit State Design Principles, and therefore it is not expected that the differences between these two codes would be significant. However, it is expected that the older code, CP114, which is based on the Allowable Stress Design principles, would differ significantly from the Eurocodes.

2.6.5. Limit State Design versus Allowable Stress Design

The current Eurocodes are based on the Limit State Design (LSD) method, whereas the historic Code of Practice CP114 was based on the Allowable Stress Design (ASD) method. It is expected that these codes will differ significantly based on their respective design methodologies due to the differences in the basic assumptions behind the behaviour of the reinforced concrete. It is important to understand the differences between these two methods in order to accurately understand whether LSD indeed results in optimised RC elements and structures.

In the ASD methodology, it was assumed that the reinforced concrete element acted as a linear elastic material and that it would fail when the elastic limit was reached. This point is indicated as F_y in Figure 7. However, the LSD method recognises that the reinforced concrete element will behave plastically after the elastic limit is reached, and therefore does not fail at point F_y.

In Figure 7, the stress versus strain relationship of reinforced concrete is shown. The first portion of the graph shows the linear stress-strain profile of the reinforced concrete element. This is the elastic region where the stress and the strain are directly proportional, and the reinforced concrete acts as a linear elastic material. When the load is removed, it will return to its original shape. Once the material reaches the elastic limit, F_y, the element starts to deform plastically. This means that when the load is removed, the element will be deformed.

The ASD method only considers a portion of the elastic region. To be conservative, a safety factor (Y) was applied to the elastic limit. The factor of safety is the ratio of the strength of the material to its permissible stress (Shekhar, 2023). This is the point F_y/Y in Figure 7. Therefore, this conservative approach leaves much of the strength of the reinforced concrete element unused.

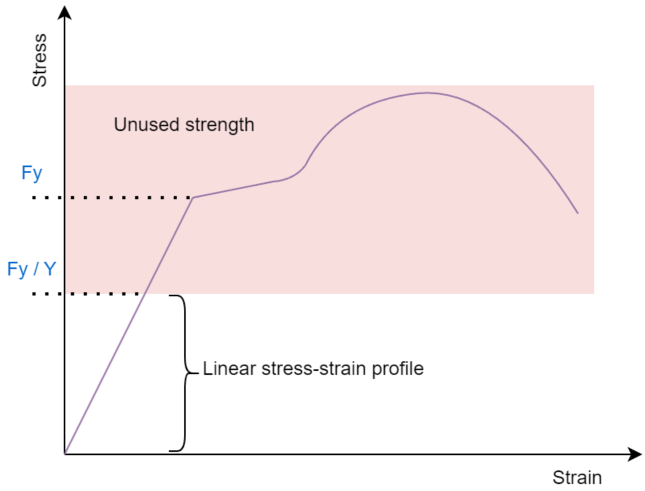

Figure 7: Stress vs Strain relationship of reinforced concrete (Shekhar, 2023)

The design and dimensions of a member design to the ASD method is controlled primarily by the allowable stress of the member. The section is designed to ensure that the axial load from the stress applied to the member, which is calculated using linear elastic theory, does not exceed a fixed allowable stress. It has been established that this method is not accurate for the design of reinforced concrete elements. This is because the method assumes that concrete and steel both obey Hooke's Law. However, only steel follows this law, and concrete does not. This is the first major limitation of this design methodology (Korabu, 2023). Further limitations of this method are as follows:

- A constant modular ratio is used in this method, which is not an accurate representation of the relationship between concrete and the reinforcing bars. In fact, there is a non-linear stress vs strain relationship between these two materials (Korabu, 2023).
- An assumption in the ASD method is that the ultimate stress is the safety limit, whereas experimental results show that it is actually governed by extreme strain (Korabu, 2023).
- Only one safety factor is applied to the load, and only the ultimate limit state is considered. This leads to a large safety factor, which may ensure a safe design, but results in bulky members with maximised cross-sections (Korabu, 2023).
- The effects of creep and shrinkage are not considered in this design method, which affects the durability of the member or structure (Korabu, 2023).
- A structure is subjected to different types of loads and therefore it is not accurate to model the failure criteria on the stresses, rather than the loads (Korabu, 2023).

As per the LSD method, the element will not fail when the elastic limit is reached, and additional stress past this point could be applied, as shown in Figure 7. Additionally, in this method, multiple factors of safety are used, instead of the one ratio used in the beforementioned method. In this method, the ultimate and serviceability limit states are considered. Adherence to the ultimate limit prevents the collapse/failure of the element. Adherence to the serviceability limits decreases crack widths, discomfort to occupants, etc. (Shekhar, 2023).

The Limit State Design (LSD) method considers and ensures safety at the ultimate load, as well as serviceability at the working load. The ultimate load and durability are both considered. The LSD method also differs from the ASD method as it considers all the different loads and the combinations thereof that act on a structure. In other words, it considers the real-life behaviours of the structure during its lifetime, right until failure. This is because the ASD method is based on Hooke's Law and elastic theory, whereas the LSD method relies rather on statistical and probabilistic approaches (Korabu, 2023).

The primary differences between the design methodologies are highlighted below:

1. The LSD method gives an economical design, whereas the ASD method does not (Korabu, 2023).
2. The ASD method applies one safety factor to the loads and does not consider different loadings. The LSD method uses and applies different safety factors to different types of loads (Korabu, 2023).
3. The ASD method assumes that the stress vs strain graph for RC sections is linear, where the LSD method uses a non-linear relationship (Korabu, 2023).
4. The ASD method does not fully utilise the strength of a member as it imposes a fixed allowable stress. The LSD methos fully utilises the potential strength of the member (Korabu, 2023).

2.7. Conclusion

Due to the ever-increasing effects of climate change, it has become essential to focus on the reduction of carbon dioxide and greenhouse gas emissions. This is particularly true for the construction sector, which accounts for approximately 37% of energy-related carbon dioxide emissions (UNEP, 2022). The construction sector which relies heavily on concrete and steel emit high concentrations of carbon dioxide during their manufacture and construction.

However, the reality is that the two primary factors considered during the construction process are the cost of the project and the safety of the structure. It is reasonable to infer that the average structural designer will design with these factors in mind and will give little thought to improving the sustainability of the structure, or to including sustainable technologies which are on the market. For example, the codes do not specify any minimum replacement of cement by SCMs, and therefore the structural designer has no obligation to use them.

It is therefore important to establish whether the design codes unintentionally result in embodied carbon savings, and to what degree. If significant savings can be ensured by using the design codes, then the overall emissions from the construction sector are reduced at a fundamental design level.

This study was used to determine the carbon dioxide savings purely from the optimization of the element dimensions and the reductions in material quantities, without considering the quality and composition of those materials. Therefore, SCMs have not been introduced into the scope of this research.

Reductions in carbon dioxide emissions through efficient structural design will directly help the construction sector in meeting the Sustainable Development Goals (SDGs) set out by the United Nations.

3. METHODOLOGY

3.1. Overview

This research project investigated the unintentional percentage savings in embodied carbon which occurred from 50 years of design code changes. The student was based in Ireland, which is part of the European Union, and uses Eurocodes as the basis for its designs. Therefore, the Eurocodes were chosen as the basis of this research project.

This study was used to determine the carbon dioxide savings purely from the optimization of the element dimensions and the reductions in material quantities, without considering the quality and composition of those materials. Therefore, SCMs were not introduced into the scope of this research.

To establish whether the evolution of the design codes unintentionally reduced the embodied carbon of a structural element, the current Eurocodes were analysed against the CP114 code which was in use more than 50 years ago. Fifty years is a significant period in which the effects of climate change have been emphasised, and carbon reductions actively sought in order to meet the climate goals set by each individual country, and to meet the Sustainable Development Goals (SDGs) set out by the United Nations.

The current design codes which were analysed were the Eurocodes listed below, as discussed in *Section 2.6*.

- Eurocode 0: Basis of structural design (EN 1990) (The European Union, 2005)
- Eurocode 1: Actions on structures (EN 1991), Part 1.1: General Actions – densities, self-weight, imposed loads for buildings (The European Union, 2002)
- Eurocode 2: Design of concrete structures (EN 1992) (The European Union, 2004)

The historic design code which was analysed was the Code of Practice CP114 *The structural use of normal reinforced concrete in buildings*, as discussed in *Section 2.6.4*.

The first structural element analysed in this research project was a simply supported beam. This type of beam was selected for analysis as it is the most commonly used statically determinate member (Dominik, 2023). It can therefore be inferred that this type of beam is present in most buildings. A statically determinate member is described as one where all forces can be calculated by hand with three equilibrium equations (Dominik, 2023). The calculations which have been carried out in this research project were conducted by first principles, to fully understand the different design methodologies of the two structural design codes. To compare the results, the calculations using both codes have been completed manually and without the use of any software. It is therefore beneficial to analyse a statically determinate member for ease of calculations.

The second structural element analysed was a column. Like the simply supported beam, a reinforced concrete column is a widely used member present in most buildings, and therefore the analysis of both elements is invaluable concerning carbon dioxide emission reductions. Once again, the calculations were done using first principles and the design methodologies from each of the chosen design codes. Columns and beams are designed to resist different types of loading, namely compression and bending, respectively. Analysing both members was essential in determining a more accurate conclusion of whether there were unintended embodied carbon reductions due to the evolution of the design code.

The Eurocodes are based on *Limit State Principles*, whereas CP114 was based on the concept of *allowable stress design*. Several structural design variables were investigated, such as design methodologies, design loads on a structure, safety factors, design coefficients, the required dimensions of the element, and the properties of the materials in use. The difference in these variables between the historic structural design codes and the current structural design codes was highlighted. This provided information on the percentage decrease in the quantity of concrete required per each code, and ultimately the carbon savings.

3.1.1. Presentation of Results

The methodologies for the design of simply supported beams and columns are presented in this chapter using the Eurocodes and CP114. The detailed analysis of one simply supported beam design and one column design are described and presented in Chapter 4. However, several simply supported beams and columns were designed, whilst changing different variables such as the lengths, concrete strengths, or the design loads. A summarised set of results for these additional iterations are presented in Chapter 5.

3.2. Simply Supported Beam Design

For this research project, a simply supported singly reinforced concrete beam as shown in Figure 8 was designed according to two different structural design codes; the current Eurocodes and the withdrawn Code of Practice CP114. *Section 3.3* outlines the design methodology according to the Eurocodes. *Section 3.4* outlines the design methodology according to CP114.

In Figure 8, a rectangular reinforced concrete beam, of length L, is supported between supports A and B. The force F is represented as a unit load at the centre of the beam. The beam is located in a concrete frame office building in Ireland.

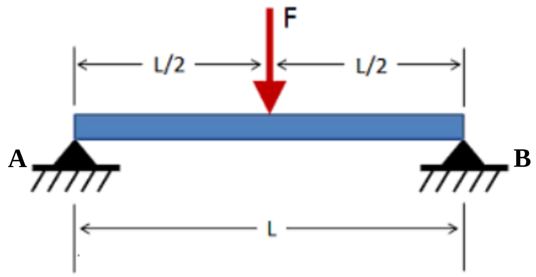

Figure 8: Elevation view of a simply supported reinforced concrete beam.

3.3. Eurocodes: Simply Supported Beam Design Methodology

3.3.1. Design Life, Exposure Class, and Concrete Strength Class

3.3.1.1. Design Life

When using the Eurocodes, consideration is first given to the design life and location of the building. The indicative design life of an element or structure is located in *EN1990* and shown in Table 4 below.

Table 4: Indicative design working life (The European Union, 2005)

Design working life category	Indicative design working life (years)	Examples
1	10	Temporary structures [(1)]
2	10 to 25	Replaceable structural parts, e.g. gantry girders, bearings
3	15 to 30	Agricultural and similar structures
4	50	Building structures and other common structures
5	100	Monumental building structures, bridges, and other civil engineering structures
(1) Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.		

3.3.1.2. Exposure Class

The Eurocodes consider the structure's exposure to the environment in which it is located. This is an important aspect in ensuring the durability of the structure under consideration. The durability is the ability of a member to last the specified design life without significant deterioration, and the exposure class of the element defines the concrete cover required by the reinforced concrete member. Sufficient cover should be provided to ensure that the reinforcement is not subject to corrosion through the formation of cracks (Civil's Guide, 2020). Various exposure classes, 1 to 6, can be found in Table 5 below.

Table 5: Exposure classes related to environmental conditions (The European Union, 2004)

Class designation	Description of the environment	Informative examples where exposure classes may occur
1 No risk of corrosion or attack		
X0	For concrete without reinforcement or embedded metal: all exposures except where there is freeze/thaw, abrasion or chemical attack For concrete with reinforcement or embedded metal: very dry	Concrete inside buildings with very low air humidity
2 Corrosion induced by carbonation		
XC1	Dry or permanently wet	Concrete inside buildings with low air humidity Concrete permanently submerged in water
XC2	Wet, rarely dry	Concrete surfaces subject to long-term water contact Many foundations
XC3	Moderate humidity	Concrete inside buildings with moderate or high air humidity External concrete sheltered from rain
XC4	Cyclic wet and dry	Concrete surfaces subject to water contact, not within exposure class XC2
3 Corrosion induced by chlorides		
XD1	Moderate humidity	Concrete surfaces exposed to airborne chlorides
XD2	Wet, rarely dry	Swimming pools Concrete components exposed to industrial waters containing chlorides
XD3	Cyclic wet and dry	Parts of bridges exposed to spray containing chlorides Pavements Car park slabs
4 Corrosion induced by chlorides from sea water		
XS1	Exposed to airborne salt but not in direct contact with sea water	Structures near to or on the coast
XS2	Permanently submerged	Parts of marine structures
XS3	Tidal, splash and spray zones	Parts of marine structures
5. Freeze/Thaw Attack		
XF1	Moderate water saturation, without de-icing agent	Vertical concrete surfaces exposed to rain and freezing
XF2	Moderate water saturation, with de-icing agent	Vertical concrete surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	High water saturation, without de-icing agents	Horizontal concrete surfaces exposed to rain and freezing
XF4	High water saturation with de-icing agents or sea water	Road and bridge decks exposed to de-icing agents Concrete surfaces exposed to direct spray containing de-icing agents and freezing Splash zone of marine structures exposed to freezing
6. Chemical attack		
XA1	Slightly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA2	Moderately aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water
XA3	Highly aggressive chemical environment according to EN 206-1, Table 2	Natural soils and ground water

3.3.1.3. Concrete Strength Class

The Eurocodes indicate which concrete strength class should be used in a design and is dependent on the intended design life of the structure or element, as well as the exposure class of the environment in which it is located. This is found in *Annex E of EN1992* and gives a clear indication of the concrete's characteristic cylinder strength (f_{ck}) as well as its characteristic cube strength ($f_{ck,cube}$), presented as C30/37, for example. The former number is the cylinder compressive strength, and the latter number is the cube compressive strength of the concrete.

3.3.2. Reinforcement Tensile Strength

The characteristic strength of high-yield steel (f_{yk}) used for the reinforcement of concrete elements has a minimum yield strength of 500 MPa.

3.3.3. Limit State Design

The Eurocodes make use of the Limit State Design methodology which uses several partial safety factors in the design. These are applied to the material properties and the loading. Refer to *Section 2.6.5* for further information on the Limit State Design method.

The design strength of a material is calculated by dividing the material strength by a partial safety factor. The design actions acting on the element are calculated by multiplying the working load by a partial safety factor.

3.3.4. Design Strength of Materials

The Limit State Design method introduces partial safety factors for the material properties of the concrete and the reinforcement, as per *Table 2.1N in EN1992*, as summarised below. These partial factors are applied to the strength of concrete and steel resulting in conservative design strengths which are used calculations (The European Union, 2004). The partial material safety factors for concrete and steel are listed below.

- Concrete partial factor $\gamma_C = 1.5$
- Steel partial factor $\gamma_S = 1.15$

To determine the design strength of the concrete (f_{cd}), the characteristic strength is divided by the partial safety factor and multiplied by a coefficient α_{cc}, as shown in Equation 2. α_{cc} is the coefficient which accounts for the long-term effects, such as creep, from when the load is applied, and how it affects the compressive strength of the concrete (The European Union, 2004). The design strength of the reinforcement (f_{yd}) is calculated similarly, as shown in Equation 3. However, no α coefficient is considered.

$$f_{cd} = \alpha_{cc}\frac{f_{ck}}{\gamma_c} \qquad \text{(Equation 2)}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \qquad \text{(Equation 3)}$$

3.3.5. Concrete Density

The normal weight concrete density (ρ_C) in the Eurocodes is 24 kN/m³, as found in *Table A1 of EN1991-1*. This value is increased by 1 kN/m³ for a normal percentage of reinforcement used in the design, resulting in an overall concrete density of 25 kN/m³.

3.3.6. Loading

Two Eurocodes were used for the determination of the loads acting on the structural element.

- Eurocode 0 (EN 1990) Basis of Structural Design
- Eurocode 1 (EN1991) Actions on Structures
    - Part 1.1: General Actions – densities, self-weight, imposed loads for buildings.

EN1990 and EN1991 introduce the concept of the permanent and imposed loads that act on structures. These are described in detail below. The permanent loads are indicated by the abbreviation G_k and these encompass the self-weight of the element as well as any permanently applied loads on the element/structure. The imposed loads are indicated by the abbreviation Q_k and these are the temporary loads that affect the structure, such as fluctuating person loads, or wind actions. These can be applied as uniformly distributed loads (q_k) or as point loads (Q_k).

Multiple temporary loads can act on a single structure at any one time. The effects of accidental actions (A), such as natural hazards or impacts from human activities like a car accident, are also described in the Eurocodes. Such loads did not form part of the scope of this research.

3.3.6.1. Permanent Loads

The permanent loads which act on a simply supported beam are the self-weight of the reinforced concrete beam, and the reinforced concrete slab resting thereon. The beam's self-weight is determined by Equation 4.

$$G_k(beam) = l \times b \times h \times \rho_c \qquad \text{(Equation 4)}$$

Where:

- l is the beam length.
- b is the beam width.
- h is the beam height.
- ρ_c is the concrete density, as determined in *Section 3.3.5*.

The supported slab also imposes a permanent load on the beam, as shown in Equation 5.

$$G_k(slab) = d_{slab} \times b_{supp} \times l_{supp} \times \rho_c \qquad \text{(Equation 5)}$$

Where:

- d_{slab} is the depth of the supported slab.
- l_{supp} is the length of the slab that is supported by the beam.
- b_{supp} is the breadth of the slab that is supported by the beam.
- ρ_c is the concrete density, as determined in *Section 3.3.5*.

Therefore, the sum of the self-weights of the beam and the supported slab gives the permanent load G_k supported by the beam.

3.3.6.2. Imposed Loads

The imposed loads applied to the beam are split into the primary ($Q_{k,1}$) and secondary ($Q_{k,2}$) imposed loads. These loadings can be found in *Eurocode 1 (EN1991): Actions on Structures*. The primary load ($Q_{k,1}$) refers to the temporary loads that arise from the usage category of the structure, as specified by *EN1991-1 Part 1.1: General Actions – Densities, self-weight, imposed loads for buildings*. The secondary load ($Q_{k,2}$) refers to the wind loads, as per *EN1991-1 Part 4: General actions – Wind actions*.

3.3.6.3. Primary Imposed Load

The Eurocodes specify the imposed loads based on the “category of use” of the structure, as per *EN1991-1-1 Table 6.1*. The corresponding imposed loads for the relevant category are then found in *EN1991-1-1 Table 6.2*.

3.3.6.4. Secondary Imposed Load

The secondary imposed load ($Q_{k,2}$) on the element is the wind load. The wind load is calculated through several steps, as detailed in *EN1991-1 Part 4: General actions – Wind actions*. However, the wind load is not considered in this research as an interior beam is considered.

3.3.7. Design Loads: Partial Safety Factors

As discussed in *Section 3.3.3*, the limit state design methodology makes use of partial safety factors for the loads as well as for the materials. The material partial factors were discussed in *Section 3.3.4.* The partial factors which are applied to the actions acting on the element can be broken up into favourable and unfavourable actions, indicated below (The European Union, 2005). Favourable actions refer to actions whose effects are reversible, such as a person walking along the beam, whereas the effects of un-favourable actions are considered irreversible, such as settling or creep, which is the long-term deformation of the beam under sustained loads.

- F: Favourable Conditions
- UN-F: Unfavourable Loads

These partial factors are further broken down in the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS), as shown in Table 6. These were found in *EN1990: Basis of Structural Design*.

Table 6: Favourable and Unfavourable Partial Factors for Actions

Action	Condition	Partial Factor	Limit State
G_k	UN-F	1.35	ULS
G_k	F	1.15	ULS
$Q_{k,1}$	UN-F	1.5	ULS
$Q_{k,1}$	F	0	ULS
$Q_{k,2}$	UN-F	1.5	ULS
$Q_{k,2}$	F	0	ULS
G_k	UN-F	1	SLS
Q_k	F	1	SLS

3.3.8. Design Loads and Combinations of Actions

The partial factors from Table 6 are not applied to the actions in isolation. The effect of combinations of actions is also considered, as per EN1990, and these are applied to the variable actions (Q_k). This combination value is represented as a product $\Psi_0 Q_k$, and it is used in the Ultimate Limit State (ULS) and the irreversible Serviceability Limit State (SLS) (The European Union, 2005).

The overall loading on the beam is taken as the maximum load from the iteration of equations based on Equation 6, where E_d represents the design load applied to the element.

$$E_d = \sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \qquad \text{(Equation 6)}$$

3.3.9. Reaction Forces and Shear Force Diagram

Figure 9 illustrates the shear force diagram for a simply supported beam. It is evident that for a simply supported beam on pin supports, the reaction force at support A is equal to the reaction force at support B, as shown in Equation 7, where $E_{d,max}$ is the maximum applied design load, as determined iteratively in *Section 3.3.8*, V is the shear force, represented by the reaction forces, and L is the length of the beam.

$$Reaction\ A = Reaction\ B = \frac{E_{d,max}}{2} \qquad \text{(Equation 7)}$$

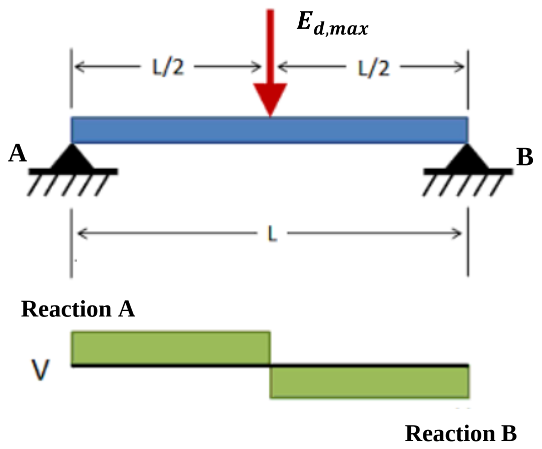

Figure 9: Shear force diagram of the simply supported beam

3.3.10. Bending Moments and Bending Moment Diagram

Figure 10 illustrates the Bending Moment Diagram (BMD) for a simply supported beam. To determine the maximum moment of the beam, Equation 8 was used.

$$M_{max} = \frac{E_{d,max} l}{4} \qquad \text{(Equation 8)}$$

Where:

- $E_{d,max}$ is the maximum applied design load, as determined iteratively in *Section 3.3.8*.
- l is the length of the beam.
- M_{max} is the maximum moment applied to the beam.

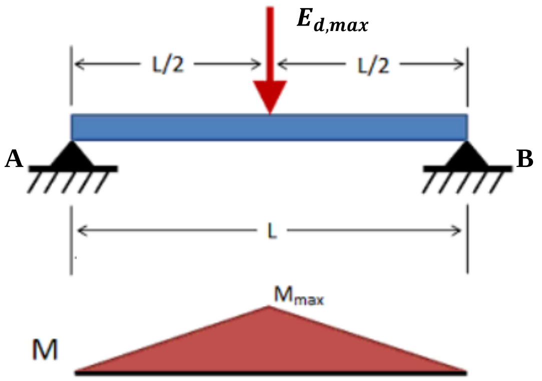

Figure 10: Bending moment diagram of the simply supported beam.

3.3.11. Concrete Cover

The concrete cover is defined as the distance from the surface of the concrete to the reinforcement (The European Union, 2004). The concrete cover provided to the reinforcement in the beam is a function of the exposure class of the element and aids in establishing a durable reinforced concrete element. The minimum concrete cover protects the steel reinforcement from corrosion and ensures adequate fire resistance. This can contribute to additional concrete required in a section. However, this is deemed as necessary in order to protect the reinforcement and prevent corrosion.

The nominal concrete cover (c_{nom}) is calculated using Equation 9.

$$c_{nom} = c_{min} + \Delta c_{dev} \qquad \text{(Equation 9)}$$

Where:
- c_{min} is the minimum concrete cover to be provided, calculated from Equation 10.
- Δc_{dev} is the allowance for deviation, as per *Section 4.5.1.3 of EN1992*.

$$c_{min} = max\{c_{min,b}; c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10mm\} \qquad \text{(Equation 10)}$$

Where:
- $c_{min,b}$ is the minimum cover due to the bond requirements and is based on the diameter of the separate reinforcement bars.
- $c_{min,dur}$ is the minimum cover due to environmental conditions which depends on the exposure class.
- $\Delta c_{dur,\gamma}$ is the additive safety element, as per *Section 4.4.1.2 of EN1992*.
- $\Delta c_{dur,st}$ is the possible cover reduction due to the use of stainless steel.
- $\Delta c_{dur,add}$ is the possible cover reduction due to additional protection measures.

3.3.12. Deflection and Minimum Depth

To ensure that the deflection of the beam is kept to a minimum, the basic span-to-effective depth ratio must be limited (The European Union, 2004). Effective depth is defined as the depth of the beam from the top surface to the midpoint of tensile reinforcement. The basic span (l) over depth (d) ratio is determined from *Table 7.4N of EN1992* as 20, for a simply supported beam with lightly stressed concrete, as shown in Equation 11.

$$\frac{l}{d} \leq 20 \qquad \text{(Equation 11)}$$

The overall limiting height of the beam is calculated as shown in Equation 12. The height of the beam is limited to avoid uncomfortable deflections. Therefore, the height of the beam should at all points exceed this value.

$$h_{limit} = d + c_{nom} + \frac{\emptyset_{rebar}}{2} \qquad \text{(Equation 12)}$$

Where:
- h_{limit} is the limiting height of the beam.
- d is the effective depth of the beam, which should be assumed to be larger than the depth calculated in Equation 11.
- c_{nom} is the nominal cover to the reinforcement, as determined in *Section 3.3.11*.
- $\emptyset_{rebar}$ is the diameter of the tension reinforcement used in the beam's cross-section.

Once the limiting height of the element was established, then the optimal height for the column could be established. This was determined by iteratively changing the height of the beam to establish at which height it would fail in shear, bending, or deflection. In other words, at which height would the beam be able to withstand the load and comply with all the checks required by the Eurocodes, such as:

- The beam must comply with the limiting span/depth ratio, as per Equation 11, as well as the in-depth deflection check described in Section 3.3.21.
- The beam must only require tension reinforcement, i.e., it is singly reinforced as per Section 3.3.13. This criterion was used as a controlled variable in this research project in order for the beams designed to the Eurocode and CP114 could be accurately compared.
- The neutral axis must fall within the required portion of the cross-sectional area of the element to minimise the deflection of the beam, as per Section 3.3.18.
- The beam will not fail in shear, as per Section 3.3.19.
- The beam will not fail in bending, as the moment which can be resisted by the beam is greater than the maximum moment applied to the beam, as per Section 3.3.20.

3.3.13. Singly Reinforced Section

The next step is to determine whether the cross-section of the beam under consideration should be singly or doubly reinforced to effectively resist the forces and moments that act on it. A singly reinforced section only requires tension reinforcement in the lower part of the cross-section. A doubly reinforced section requires both tensile and compressive reinforcement, and the compressive reinforcement is placed in the upper portion of the cross-section (The European Union, 2004).

To determine if a beam is singly or doubly reinforced, it is important to understand how the stress and strain act over the cross-section of the beam. Figure 11 shows the stress and strain distributions over a rectangular cross-section (The European Union, 2004). A balanced section is where the compression forces in the compression block balance with the tension forces in the reinforcement.

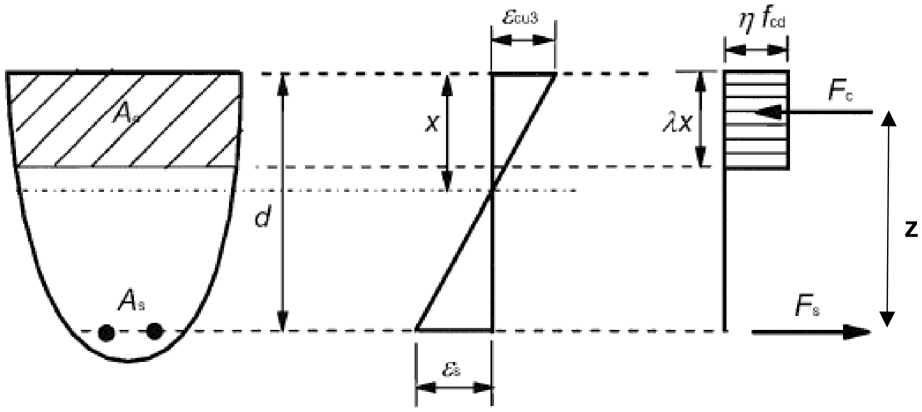

Figure 11: Strain and stress acting over the cross-section of a reinforced concrete beam (The European Union, 2004)

Where:
- F_s represents the tensile forces in the steel reinforcement.
- F_c represents the compressive forces in the concrete.
- A_c is the compression block area at the top of the cross-section. This corresponds to the depth of the stress block as shown on the right-hand side (The European Union, 2004).
- A_s is the area of the tension steel reinforcement.
- d is the effective depth to the midpoint of the tension steel reinforcement.

- x is the depth to the neutral axis of the cross-section.
- λ defines the effective height of the compressive zone and has a value of 0.8 for concrete strength classes less than 50 MPa.
- ε_s is the strain in the tensile reinforcement.
- ε_{cu3} is the ultimate compressive strain shown as an absolute value and has a value of 0.0035 as per *EN1992* and represented in Figure 12.
- f_{cd} is the design value of the concrete compressive strength, as per *Section 3.3.4*.
- η defines the effective strength and has a value of 1.0 for concrete strength classes less than 50 MPa.
- z is the lever arm, as per *Section 3.3.14*.

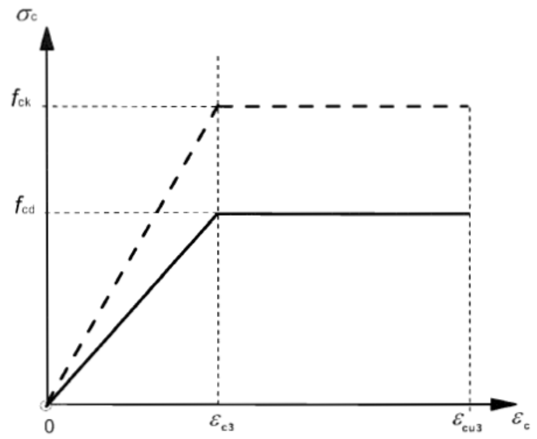

Figure 12: Bi-Linear Stress versus Strain Relationship (The European Union, 2004)

To determine whether the section is to be singly or doubly reinforced, the limiting factor K is determined according to Equation 13. If K < 0.167, then only tensile reinforcement is required. However, if K ≥ 0.617, then compression steel is also required (The European Union, 2004).

$$K = \frac{M_{max}}{bd^2 f_{ck}} \leq 0.167 \qquad \text{(Equation 13)}$$

Where:
- M_{max} is the maximum bending moment, as per *Section 3.3.10*.
- b is the breadth of the beam.
- d is the effective depth of the beam.
- f_{ck} is the characteristic concrete strength, as per *Section 3.3.1*.

3.3.14. Lever Arm

The lever arm (z) is the vertical distance between the force in the steel (F_s) and the force in the compression block (F_c), as shown in Figure 11 (The European Union, 2004). This is calculated in Equation 14.

$$z = d\left[0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right] \qquad \text{(Equation 14)}$$

Where:

- z is the lever arm.
- d is the effective depth of the reinforcing steel.
- K is the limiting factor determined in *Section 3.3.13*.

At this point, a check must occur to ensure compliance with the code's upper and lower limits. The code specifies that lever arm z should have a maximum value of 0.95d. This check ensures that the lever arm is less than 95% of the effective depth. This is done to ensure that the compression stress block is not too thin, as shown in Figure 11. Equation 15 ensures that the lever arm of the section is less than 95% of the effective depth (The European Union, 2004). If the lever arm is determined to be more than this value, then a new lever arm should be used in further calculations, as the value of 0.95d.

$$z < 0.95d \qquad \text{(Equation 15)}$$

3.3.15. Required Reinforcement

The required area of reinforcement in the beam cross-section is calculated using Equation 16. This required area of reinforcement will ensure that the beam can withstand the forces and moments applied to it.

$$A_{s,req} = \frac{M_{max}}{0.87 f_{yk} z} \qquad \text{(Equation 16)}$$

Where:

- $A_{s,req}$ is the cross-sectional area of steel reinforcement required in the beam.
- M_{max} is the maximum bending moment applied to the beam, as per *Section 3.3.10*.
- f_{yk} is the yield strength of the steel reinforcement, as per *Section 3.3.2*.
- z is the lever arm of the beam's cross-section, as per *Section 3.3.14*.

3.3.16. Provided Reinforcement

Assuming that no compression reinforcement required in the cross-section. However, the tension reinforcement provided in the section should be more than or equal to the quantity of the required reinforcement, calculated in *Section 3.3.15*. The number of reinforcement bars and their cross-sectional area are determined using Table 7 (Prasad, 2023).

Table 7: Reinforcement Area Chart (StructX, 2022)

	Cross-sectional Area (mm²)								
	Bar Diameter (mm)								
No. of Bars	**10**	**12**	**16**	**20**	**24**	**28**	**32**	**36**	**40**
1	78	113	201	314	452	616	804	1020	1260
2	156	226	402	628	904	1232	1608	2040	2520
3	234	339	603	942	1356	1848	2412	3060	3780
4	312	452	804	1256	1808	2464	3216	4080	5040
5	390	565	1005	1570	2260	3080	4020	5100	6300
6	468	678	1206	1884	2712	3696	4824	6120	7560
7	546	791	1407	2198	3164	4312	5628	7140	8820
8	624	904	1608	2512	3616	4928	6432	8160	10080
9	702	1017	1809	2826	4068	5544	7236	9180	11340
10	780	1130	2010	3140	4520	6160	8040	10200	12600
Min. Hole (mm)	12	15	20	25	29	34	39	44	49

3.3.17. Minimum and Maximum Reinforcement

It is important to ensure that the reinforcement which is provided in the cross-section surpasses the minimum reinforcement required for the section, as shown in Equation 17 while being less than the maximum reinforcement required, as shown in Equation 18. This ensures that the section is not over-reinforced.

$$A_{s,prov} > A_{s,min} = 0.13\%bd \qquad \text{(Equation 17)}$$

$$A_{s,prov} < A_{s,max} = 4\%bd \qquad \text{(Equation 18)}$$

3.3.18. Cross-Sectional Neutral Axis

The neutral axis of the beam cross-section is determined through the equilibrium of the compression force of the concrete and the tensile force of the steel reinforcement, as shown in Figure 11 (The European Union, 2004). This neutral axis is calculated per Equations 19 and 20. It should be noted that this is calculated in order to assess the deflection of the beam.

$$F_c = F_s \qquad \text{(Equation 19)}$$

$$\eta f_{cd} b \lambda x = f_{yd} A_s \qquad \text{(Equation 20)}$$

Where:
- F_c is the compressive forces in the concrete.
- F_s is the tensile force in the steel reinforcement.
- η defines the effective strength and has a value of 1.0 for concrete strength classes less than 50 MPa.
- f_{cd} is the design value of the concrete compressive strength, as per *Section 3.3.4*.
- b is the breadth of the beam.
- λ defines the effective height of the compressive zone and has a value of 0.8 for concrete strength classes less than 50MPa.
- x is the depth to the neutral axis of the cross-section.
- f_{yd} is the design tensile strength of the reinforcement as per *Section 3.3.4*.
- A_s is the provided area of the tension steel reinforcement, as per *Section 3.3.16*.

A factor that was used to establish whether compressive reinforcement was required was the limiting x/d ratio, as defined in Equation 21. To ensure that the tension reinforcement yields at ULS, leading to a ductile failure, limit the x/d ratio to 0.617.

$$\frac{x}{d} < \frac{1}{\frac{f_{yk}}{\gamma_s E_s \varepsilon_{cu3}} + 1} = 0.617 \qquad \text{(Equation 21)}$$

Where:
- x is the depth to the cross-section neutral axis determined in Equation 20.
- d is the effective depth of the tension reinforcement.
- f_{yk} is the characteristic strength of the reinforcement, as per *Section 3.3.2*.
- γ_s is the partial factor for steel, as per *Section 3.3.4*.
- E_s is the design value of the modulus of elasticity of reinforcing steel, taken as 200GPa.
- ε_{cu3} is the shortening concrete strain taken as 0.0035, as described in *Section 3.3.13*.

3.3.19. Check for Shear Resistance

To ensure that the section can accommodate the maximum shear force applied to the beam, the shear resistance of the section should be checked, as per *Equation 22*.

$$V_{Rd,max} = 0.124bd\left(1 - \frac{f_{ck}}{250}\right) f_{ck} \qquad \text{(Equation 22)}$$

Where:
- $V_{Rd,max}$ is the maximum shear force that the section can resist.
- b is the breadth of the beam.
- d is the effective depth of the reinforcement.
- f_{ck} is the characteristic strength of the concrete, as per *Section 3.3.1*.

3.3.20. Bending Moment of Resistance

It is important to establish whether the beam section can accommodate the maximum bending moment applied to the beam. Therefore, the maximum moment that can be resisted by the beam should be greater than the maximum moment applied to the beam (The European Union, 2004). The bending moment of resistance is calculated as per Equation 23.

$$M_r = A_s f_{yd} z \qquad \text{(Equation 23)}$$

Where:
- M_r is the bending moment which can be resisted by the beam’s cross-section.
- A_s is the cross-sectional area of the reinforcement provided in the beam, as per *Section 3.3.16*.
- f_{yd} is the design strength of the reinforcement, as per *Section 3.3.4*.
- z is the lever arm of the beam's cross-section, as determined in *Section 3.3.14*.

3.3.21. Deflection Check

Deflections do not need to be checked explicitly if they conform to the limiting span/depth ratios in Equations 24 or 25, as per *Section 7.4.2. in EN1992* (The European Union, 2004). Conformance with the limits will ensure that the element does not deflect excessively in the SLS.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \; if \; \rho \leq \rho_0 \qquad \text{(Equation 24)}$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho - \rho\prime}\right) + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho\prime}{\rho_0}}\right] \; if \; \rho > \rho_0 \qquad \text{(Equation 25)}$$

Where:

- $\frac{l}{d}$ is the limiting span/depth ratio.
- ρ_0 is the reference reinforcement ratio, taken as $10^{-3}\sqrt{f_{ck}}$.
- ρ is the required tension reinforcement ratio at the middle of the beam to resist the moment due to the design loads. Therefore, $\rho = \frac{A_{s,req}}{bd}$.
- ρ' is the required compression reinforcement ratio at the middle of the beam to resist the moment due to the design loads. This is zero if no compression reinforcement is provided.
- K is a factor that accounts for different structural systems and is equal to 1 for a simply supported beam, as per *Table 7.4N in EN1992* (The European Union, 2004).
- f_{ck} is the characteristic strength of the concrete, as per *Section 3.3.1*.

3.4. Code of Practice CP114: Simply Supported Beam Design Methodology

3.4.1. Concrete Strength Class

The concrete strength class used in the Code of Practice CP114 (Codes of Practice Committee for Building , 1969) is determined through one of two methods of concrete mix design.

- Method 1: Nominal concrete mixes, whereby the general proportions of ingredients are provided by the code, and the laboratory and field tests have already been performed on these standard mixes.
- Method 2: Designed concrete mixes, whereby the proportions of the constituents are not standardised, and testing by the designer is required.

For this research project, Method 1 was used as all relevant tests and material factors have already been determined and are specified in the code. It was assumed that the nominal design concrete mixes would have been predominantly used in practice because the laboratory and field tests had already been completed, requiring less work by the designer.

Three nominal concrete mixes are specified in CP114, as shown in Table 8, which is a representation of *Table 1 in CP114*. This table represents the cube compressive strength f_{ck} at 28 days. As per the code, the "works tests" should be used for the nominal mixes. For intermediate mix designs, the "preliminary works" values should be used.

Table 8: Nominal Concrete Mix Proportions as per CP114 (Codes of Practice Committee for Building , 1969)

Mix Proportions	Cubic meters of aggregate per 50kg of cement		Cube strength within 28 days after mixing		Alternative cube strength within 7 days after mixing	
	Fine	Coarse	Preliminary Test (N/mm²)	Works Test (N/mm²)	Preliminary Test (N/mm²)	Works Test (N/mm²)
1:1:2	0.035	0.07	40	30	26.7	20
1:1.5:3	0.05	0.10	31	25.5	22.7	17
1:2:4	0.07	0.14	28	21	18.7	14

3.4.2. Reinforcement Tensile Strength

There were three types of reinforcement primarily in use in the middle of the 1900's. These consisted of mild steel bars, cold-worked deformed bars, and hot-rolled deformed bars. A designer could choose among these three, as CP114 refers to the following British Standards:

- B.S. 4449 Hot rolled steel bars for the reinforcement of concrete.
- B.S. 4461 Cold worked steel bars for the reinforcement of concrete.

The above standards have been revised numerous times over the decades, however, at the time of use of CP114, the standards from 1969 were in use. B.S. 4449-1969 covers plain round mild steel bars with a yield strength of 250 MPa, and hot rolled deformed bars with a yield strength of 410 MPa. *B.S. 4461-1969* covers cold-worked deformed bars with a yield strength of 460MPa for bars up to 16mm in diameter (The Concrete Society, 2006).

For this research, B.S.4449-1969 was used to compare the effects of the mild steel bars versus the hot rolled deformed bars. A summary of the reinforcement yield strengths is shown below. However, in order to accurately assess whether there were embodied carbon reductions from the quantities of the reinforcement present, it was assumed that the embodied carbon of the mild reinforcement and the high yield reinforcement was the same.

- Mild reinforcement yield strength f_y= 250 MPa
- Hot rolled deformed reinforcement yield strength f_y = 410 MPa

3.4.3. Concrete Density

CP114 specifies the concrete density (ρ_c) in *Clause 302* as 2400 kg/m³ for a general rate of 2% reinforcement.

It should be noted that 2% is a significantly higher rate of reinforcement compared to the Eurocodes, which specify a minimum reinforcement of 0.13%, as per Section 3.3.17.

3.4.4. Load Factor Method

In the load factor method, a safety factor is applied to the design loads. This is done to conservatively design the element and to ensure that failure is not reached if the design loads fluctuate. However, the Limit State Design method applies a partial safety factor to the materials used, whereas CP114 does not apply any safety factors to the concrete or the reinforcement. The load factor of 1.8 used in CP114, and it is defined as per Equation 26.

$$Load\ Factor\ LF = \frac{Ultimate\ strength\ of\ the\ beam}{Working\ load} = 1.8 \qquad \text{(Equation 26)}$$

3.4.5. Loading

The loads applied to the beam are broken into dead loads and live loads. The dead loads refer to the loads that are permanently applied to the element, including the self-weight of the element. The live loads refer to the loads that are temporarily applied to the element, such as general office loads and wind loads.

3.4.5.1. Dead Loads

The self-weights of the beam and the supported slab are determined by multiplying the concrete volume by the density of the concrete, as shown in Equations 27 and 28. Therefore, the dead load acting on the beam is the sum of the self-weights of the beam and the supported slab.

$$DL(beam) = l \times b \times h \times \rho_c \qquad \text{(Equation 27)}$$

$$DL(slab) = d_{slab} \times b_{supp} \times l_{supp} \times \rho_c \qquad \text{(Equation 28)}$$

Where:
- l is the length of the beam.
- b is the width of the beam.
- h is the height of the beam.
- ρ_c is the concrete density, as per *Section 3.4.3*.
- d_{slab} is the depth of the slab.
- b_{supp} is the breadth of the supported slab.
- l_{supp} is the length of the supported slab.

3.4.5.2. Live Loads

Code of Practice CP114 refers to *Code of Practice CP3 Basic data for the design of buildings, Chapter V: Loading* to determine the live loads which act on a structure. However, due to being recalled, *CP3 Part 1: Dead and Live Loads* could not be located by the student, despite reaching out to the South African Cement and Concrete Institute, which also did not have a hard copy or electronic copy of the code. Therefore, as per the advice of the South African Cement and Concrete Institute, the *British Standard B.S.9399 Part 1: Loading for Buildings* was used in its place as the CP3 code was shortly proceeded by this code.

3.4.5.3. Loading Summary

The overall loading according to CP114 is the sum of the dead loads and the live loads, as shown in Equation 29, which is then multiplied by the Load Factor, as shown in Equation 30.

$$P = DL + LL \qquad \text{(Equation 29)}$$

$$P_{LF} = 1.8 \times P \qquad \text{(Equation 30)}$$

It should be noted here that the Load Factor used in this code is higher than that from the Eurocode. Even if the partial safety factors from the Eurocode are averaged together. The partial safety factors are found in Section 3.3.7.

3.4.6. Reaction Forces and Shear Force Diagram

The reaction forces are calculated in the same manner as *Section 3.3.9* using the shear force diagram shown in Figure 9, using Equation 31.

$$Reaction\ A = Reaction\ B = \frac{P_{LF}}{2} \qquad \text{(Equation 31)}$$

Where:

- P_{LF} is the design load applied to the beam, determined in *Section 3.4.5*.

3.4.7. Bending Moments and Bending Moment Diagram

The maximum bending moment is calculated in the same manner as *Section 3.3.10* using the Bending Moment Diagram from Figure 10, using Equation 32.

$$M_{max} = \frac{P_{LF} l}{4} \qquad \text{(Equation 32)}$$

Where:

- M_{max} is the maximum bending moment acting on the beam.
- P_{LF} is the design load applied to the beam, determined in *Section 3.4.5*.
- l is the length of the beam.

3.4.8. Concrete Cover

Clause 307 in CP114 states that the minimum cover in a reinforced concrete element should not be less than 25mm, nor less than two times the diameter of the reinforcing bar.

3.4.9. Deflection and Minimum Depth

The minimum depth of the beam is governed by the span-over-depth ratio. The permissible span over depth ratio as per *Table 13* of CP114 is 20 for simply supported beams. Adhering to this span-over-depth ratio limits the deflection of the beam.

3.4.10. Permissible Stresses

The stresses which are permissible in reinforced concrete for nominal mix designs are specified in *Clauses 303 and 304* of CP114, for concrete and reinforcement respectively, as shown in Table 9. The maximum design stresses in the beam should be less than the permissible stresses specified in the code to ensure that the beam does not fail. The design stresses in the beam are controlled by the dimensions of the beam. The ultimate dimensions allow the permissible stresses in the concrete to be maintained and not surpassed. The maximum moment should also remain less than the bending moment of resistance, described in *Section 3.4.12*.

Table 9: Permissible Material Stresses as per CP114

Permissible Stresses in Concrete	
Direct compressive strength	ρ_{oc}
Compressive strength due to bending	ρ_{ob}
Permissible Stresses in Reinforcement	
Tensile stress in mild steel bars (Φ ≤40mm)	ρ_{st}
Compressive stress in mild steel bars (Φ ≤40mm)	ρ_{sc}
Tensile stress in deformed, high-yield bars (Φ ≤20mm)	ρ_{st}
Compressive stress in deformed, high-yield bars (Φ ≤20mm)	ρ_{sc}

Certain modifications to the permissible stresses in Table 9 are allowable. The first modification is the Age Factor for permissible compressive stresses in concrete, as per *CP114* (Codes of Practice Committee for Building , 1969). If the element is loaded after one month, then the Age Factor is taken as 1.0 and does not allow for any increases to the permissible stresses. The Age Factor increases as the loading age of the member increases.

The second modification to the permissible stresses relates to slender beams. Here the slenderness ratio is used, as shown in Equation 33. If the slenderness ratio is larger than the limiting factor of 30, then the permissible stresses are reduced (Codes of Practice Committee for Building , 1969).

$$\frac{l}{b} \leq 30 \qquad \text{(Equation 33)}$$

3.4.11. Required Reinforcement

The area of the tensile reinforcement required in the cross-section of the element to ensure a balanced section is determined from Equation 34. The required reinforcement should be more than 0.15% of the cross-sectional area of the concrete.

$$\frac{A_{st,bal}}{bd_1} = \frac{1}{3}\frac{\rho_{ob}}{\rho_{st}} \qquad \text{(Equation 34)}$$

Where:
- $A_{st,bal}$ is the area of reinforcement required to ensure a balanced cross-section.
- b is the breadth of the beam.

- d_1 is the effective depth of the tensile reinforcement in the beam.
- ρ_{ob} is the permissible compressive strength due to bending, as per *Section 3.4.10*.
- ρ_{st} is the permissible tensile stress in the reinforcement, as per *Section 3.4.10*.

The tension reinforcement provided in the section should be more than or equal to the required reinforcement calculated in Equation 34. The number of reinforcement bars and their cross-sectional area are determined from Table 7.

3.4.12. Resistance Moments

The bending moment value which should be resisted by the beam depends on Equations 35 and 36. The maximum resistance moment that can be sustained by the beam is the lesser of the two answers from these equations. The maximum bending moment that acts on the beam should be less than the resistance moment (Codes of Practice Committee for Building , 1969).

$$M_{r1} = A_{st}\rho_{st}l_a \qquad \text{(Equation 35)}$$

$$M_{r2} = \frac{\rho_{ob}}{4}bd_1{}^2 \qquad \text{(Equation 36)}$$

Where:
- A_{st} is the area of tensile reinforcement, as determined in *Section 3.4.11*.
- ρ_{st} is the permissible tensile stress in the reinforcement, as per *Section 3.4.10*.
- ρ_{ob} is the compression stress due to bending, as per *Section 3.4.10*.
- b is the width of the beam.
- d_1 is the effective depth of the tensile reinforcement in the beam.
- l_a is the lever arm, as per Equation 37.

$$l_a = d_1 - \frac{3A_{st}\rho_{st}}{4b\rho_{ob}} \qquad \text{(Equation 37)}$$

3.4.13. Working Stresses

To determine whether the compressive stress of the concrete (σ_c) is suitably lower than the permissible compressive stress of the concrete due to bending (ρ_{ob}), Equation 38 was used.

$$\sigma_c = \frac{P_{LF}}{bh} \qquad \text{(Equation 38)}$$

Where:
- σ_c is the stress in the concrete.
- P_{LF} is the design force applied to the beam, as per *Section 3.4.5*.
- b is the breadth of the beam.
- h is the height of the beam.

The same concept can be applied to the reinforcement, to ensure that the imposed stresses (σ_s) are less than the permissible tensile stresses (ρ_{st}) allowed by the code. This is shown in Equation 39.

$$\sigma_s = \frac{P_{LF}}{A_{st}} \qquad \text{(Equation 39)}$$

Where:

- σ_s is the stress in the reinforcement.
- P_{LF} is the design force applied to the beam, as per *Section 3.4.5*.
- A_{st} is the area of tensile reinforcement, as determined in *Section 3.4.11*.

3.5. Column Design

A reinforced concrete column situated in a reinforced concrete frame structure, shown in Figure 13, was designed according to the two different structural design codes; the current Eurocodes and the withdrawn Code of Practice CP114. *Section 3.6* describes the design methodology according to the Eurocodes. Section 3.7 describes the design methodology according to CP114. The column under consideration is an internal column in a three-story braced structure.

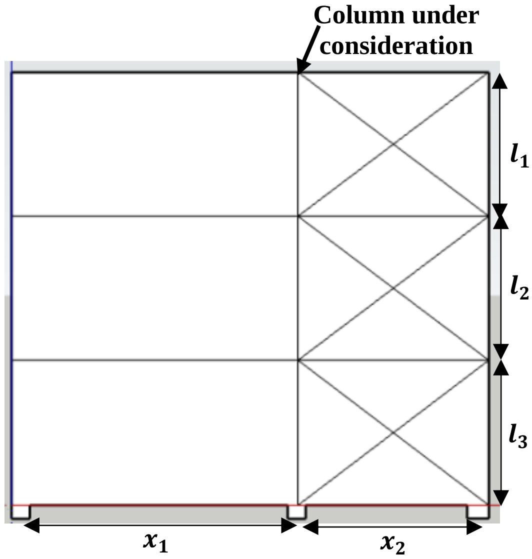

Figure 13: Interior reinforced concrete column in a braced structure

Columns are compression members that transfer loads from a structure to the foundations. Columns are classified as short or slender depending on their slenderness ratio. This affects the mode of failure, which can be by crushing, buckling, or shearing (Obinna, 2020).

The methodology for the column design detailed in the following section only considers a column in a braced structure, as shown in Figure 13. In a braced structure, the lateral loads are resisted by the bracing, as shown on the right-hand side of the frame. Therefore, there are axial forces that act on the columns and bending moments at the ends of a column caused by these vertical actions (Obinna, 2020).

3.6. Eurocodes: Column Design Methodology

3.6.1. Material Properties

The concrete and steel reinforcement material properties established in the simply supported beam design were also applicable to the following column design. A summary of these values is presented in Table 10, and a reference to the corresponding sections is required for further information.

Table 10: Concrete and steel reinforcement material properties

Material Property	Symbol	Section Reference
Concrete characteristic cylinder strength	f_{ck}	3.3.1
Concrete characteristic cube strength	$f_{ck,cube}$	3.3.1
Characteristic strength of high-yield steel reinforcement	f_{yk}	3.3.2
Concrete partial factor	$\gamma_C = 1.5$	3.3.4
Concrete density	ρ_C	3.3.3

3.6.2. Loading

Eurocode 0 (EN 1990) Basis of Structural Design (The European Union, 2005) and *Eurocode 1 (EN1991) Actions on Structures* (The European Union, 2002) were used for the determination of the loads acting on the structural element. For further information, see *Section 3.3.6*. Column design is governed by the ULS to avoid deflections and cracking. The design axial load at ULS is used in the determination of slenderness and the required reinforcement area (Obinna, 2020).

The permanent loads (G_k) encompass the self-weight of the beams and the supported slab as well as any permanently applied loads on the element/structure, such as any permanent fixtures in the building like cupboards or servers. However, these weren't included in the loading for this research project. The imposed loads (Q_k) are the temporary loads that affect the structure, such as fluctuating person loads. These can be applied as uniformly distributed loads (q_k) or as point loads (Q_k).

3.6.2.1. *Permanent Loads*

The permanent loads applied to a column are the self-weight of the reinforced concrete beams, and the reinforced concrete slab resting thereon, as well as the weight of the column. The beam's self-weight is determined by Equation 40.

$$G_k(beam) = \left(\frac{l_1+l_2}{2}\right) \times h \times \rho_c \qquad \text{(Equation 40)}$$

Where:
- $G_k(beam)$ is the self-weight of the beam, calculated as a force per metre length of the beams in the frame.
- l_1 is the beam span on the one side of the column.
- l_2 is the beam span on the other side of the column.
- h is the beam height.
- ρ_c is the concrete density, as per *Section 3.6.1*.

The supported slab also imposes a permanent load on the column, as shown in Equation 41.

$$G_k(slab) = \left(\frac{l_1+l_2}{2}\right) \times d_{slab} \times \rho_c \qquad \text{(Equation 41)}$$

Where:
- $G_k(slab)$ is the self-weight of the slab, calculated as a force per metre length of the beams in the frame.
- l_1 is the beam span on the one side of the column.
- l_2 is the beam span on the other side of the column.
- d_{slab} is the depth of the supported slab.
- ρ_c is the concrete density, as per *Section 3.6.1*.

Therefore, the sum of the self-weights of the beams and the supported slab gives the permanent load G_k supported by the column.

3.6.2.2. *Imposed Loads*

The imposed load (Q_k) refers to the temporary loads that arise from the usage category of the structure, as specified by *EN1991-1 Part 1.1: General Actions – Densities, self-weight,* *imposed loads for buildings, Table 6.1*. The corresponding imposed loads for the relevant category are then found in *EN1991-1-1 Table 6.2* (The European Union, 2002).

3.6.3. Design Loads: Partial Safety Factors

The partial factors (γ) which are applied to the actions acting on the element can be broken up into favourable and unfavourable actions, as described in *Section 3.3.7*. (The European Union, 2005). The overall loading on the column is taken as the maximum load from the iteration of equations based on Equation 42, where E_d represents the design load applied to the element.

$$E_d = \sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} \qquad \text{(Equation 42)}$$

Where:
- E_d is the design load applied to the column.
- $\gamma_{G,j}$ is the partial factor applied to the permanent load, as listed in *Section 3.3.7*.
- $G_{k,j}$ is the permanent load.
- $\gamma_{Q,1}$ is the partial factor applied to the temporary load, as listed in *Section 3.3.7*.
- $Q_{k,1}$ is the temporary load.

To calculate the design load as an axial load applied to the column, E_d should be multiplied by half the length of each of the beams that it supports in the frame.

In column design, it is important to consider the critical load which is the sum of the loading on the column of all the floors above it. Therefore, the ground floor column sustains the maximum axial load from all the floors above it.

3.6.4. Short versus Slender Columns

In the Eurocodes, columns are classified as either "short" or "slender". The key differences between the two are highlighted in Table 11. It is beneficial to ensure that a column is short, rather than slender, so that second-order effects can be ignored. If a column has a slenderness ratio below a given limit, then it is designed to simply resist axial actions and moments. If the slenderness of the column exceeds the prescribed limit, then the additional second-order moments must be accounted for (Obinna, 2020). Therefore, the slenderness ratio of a column was checked, to determine if the column is short or slender.

Table 11: Key differences between short and slender columns (Obinna, 2020)

Short Columns	Slender Columns
The concrete area to effective length ratio is small.	The concrete area to effective length ratio is larger than that of short columns.
Less likely to buckle.	Likely to buckle.
The mode of failure is crushing.	The mode of failure is buckling.
Slenderness ratio: $\lambda < \lambda_{lim}$	Slenderness ratio: $\lambda \geq \lambda_{lim}$

3.6.4.1. Area Moment of Inertia

The initial step in establishing whether a column is short or slender is to determine the area moment of inertia. The area moment of inertia is a parameter that quantifies how much resistance a cross-section has towards bending (The Efficient Engineer, 2023).

The area moment of inertia is denoted by the symbol I and for a rectangular cross-section is calculated using Equation 43, as shown in Figure 14.

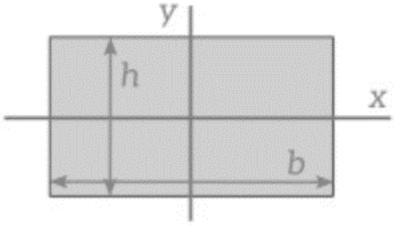

Figure 14: Cross-section of a column (The Efficient Engineer, 2023)

$$I_{axis} = \frac{bh^3}{12} \qquad \text{(Equation 43)}$$

Where:

- I_{axis} is the area moment of inertia of an element's cross-section. This is dependent on the axis around which the bending is considered.
- b is the breadth of the member in the direction parallel to the axis under consideration.
- h is the height of the member in the direction perpendicular to the axis under consideration.

3.6.4.2. Stiffness

Stiffness is the ability of an element to resist deflection or deformation by an applied force (The Engineering Toolbox, 2008).

Stiffness is calculated using Equation 44. However, for a system of members, for example, columns and beams, the relative stiffness between the members at the point where they meet can be calculated using Equation 45. If the members are of the same material, then the elastic modulus (E) is cancelled out in the fraction.

$$k_1 = \frac{EI}{L} \qquad \text{(Equation 44)}$$

$$k_1 = \frac{k_{col}}{\sum k_{beam}} = \frac{(^{EI}/_{L})_{col}}{2\sum(^{EI}/_{L})_{beam}} \qquad \text{(Equation 45)}$$

Where:

- k_1 is the stiffness of the element at one of its endpoints.
- E is the elastic modulus of the element.
- I is the area moment of inertia, as calculated in *Section 3.6.4.1*.
- L is the length of the element.

3.6.4.3. Radius of Gyration

The radius of gyration is used to quantify the elastic stability of an element's cross-section against buckling and is dependent on the axis under consideration (Anwar & Najam, 2017). For a rectangular cross-section, Equation 46 is used to determine the radius of gyration.

$$i = \sqrt{\frac{I}{A}} \qquad \text{(Equation 46)}$$

Where:

- i is the radius of gyration.
- I is the area moment of inertia around the axis under consideration, as per *Section 3.6.4.1*.
- A is the cross-sectional area of the element.

3.6.4.4. Effective Column Length

Clause 5.8.3.2 in EN1992 defines the effective length of a compression member in a regular braced frame, as shown in Equation 47, (The European Union, 2004).

$$l_0 = 0.5l\sqrt{\left(1+\frac{k_1}{0.45+k_1}\right)\left(1+\frac{k_2}{0.45+k_2}\right)} \qquad \text{(Equation 47)}$$

Where:

- l_0 is the effective length of the column.
- l is the length of the column.
- k_1, k_2 are the relative stiffnesses at ends 1 and 2 respectively, as per *Section 3.6.4.2*.

3.6.4.5. Slenderness Ratio

The slenderness ratio from *Clause 5.8.3.2 in EN1992* is defined in Equation 48.

$$\lambda = \frac{l_0}{i} \qquad \text{(Equation 48)}$$

Where:

- λ is the slenderness ratio of the column for the axis under consideration (The European Union, 2004).
- l_0 is the column's effective length, as determined in *Section 3.6.4.4*.
- i is the radius of gyration of the column, as determined in *Section 3.6.4.3*.

3.6.4.6. Slenderness Limit

The slenderness limit is described in *Clause 5.8.3.1 in EN1992* and determined using Equations 49 to 54. These equations can be simplified to the expression in Equation 55 if all the recommended values prescribed in the clause are used.

$$\lambda_{lim} = 20 \times A \times B \times C/\sqrt{n} \qquad \text{(Equation 49)}$$

$$A = 1/(1+0.2\varphi_{ef}) \qquad \text{(Equation 50)}$$

$$B = \sqrt{1+2\omega} \qquad \text{(Equation 51)}$$

$$C = 1.7 - r_m \qquad \text{(Equation 52)}$$

$$r_m = \frac{M_{01}}{M_{02}} \qquad \text{(Equation 53)}$$

$$n = \frac{N_{Ed}}{A_c f_{cd}} \qquad \text{(Equation 54)}$$

Where:

- φ_{ef} is the effective creep ratio.
- A is shown in Equation 50 and is taken as 0.7 if φ_{ef} is unknown.

- ω is the mechanical reinforcement ratio.
- B is shown in Equation 51 and is taken as 1.1 if ω is unknown.
- r_m is the moment ratio of the element's end moments, as shown in Equation 53. For braced members where the column is in tension on the same side, r_m is taken as 0.
- C is shown in Equation 52 and is taken as 0.7 if r_m is unknown.
- N_{Ed} is the design axial load at ULS, as per *Section 3.6.3*.
- A_c is the area of the column's cross-section.
- f_{cd} is the design concrete strength, as per *Section 3.3.4*.

$$Simplified\ \lambda_{lim} = \frac{26.2}{\sqrt{\frac{N_{Ed}}{A_c f_{cd}}}} \quad \text{(Equation 55)}$$

3.6.5. Column Reinforcement

The cross-sectional area of the reinforcement required in the columns can be estimated from the rectangular column charts which have been created based on EN1992. An example of such a chart is shown in Figure 15. These charts may only be used for reinforcement placed symmetrically in a column cross-section and concentrated in the corners. These charts may also only be used if the characteristic concrete strength (f_{ck}) is less than 50MPa, and the characteristic strength of steel reinforcement (f_{yk}) is less than 500MPa.

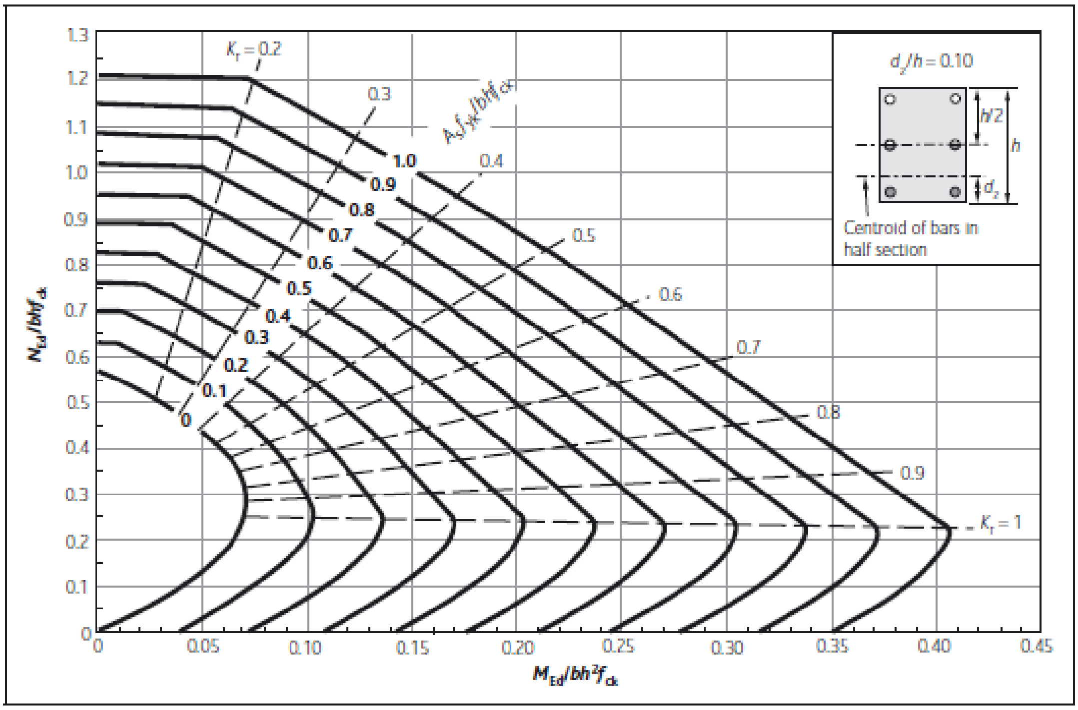

Figure 15: Reinforcement chart for Eurocode Column Designs

The correct chart must be chosen based on the d_2/h ratio, where d_2 is the depth to the centroid of the bars from the surface of the concrete and h is the height of the member in the direction perpendicular to the axis under consideration.

To use the chart shown in Figure 15, the applied axial load (N_{Ed}) and the applied bending moment (M_{Ed}) are required to determine the cross-sectional area of reinforcement (A_s). These values are based on Equations 56 to 62 below.

$$k_{col} = \left({}^{I}/_{L}\right)_{col} \quad \text{(Equation 56)}$$

$$k_{beam,1} = \left({}^{I}/_{L}\right)_{beam,1} \quad \text{(Equation 57)}$$

$$k_{beam,2} = \left({}^{I}/_{L}\right)_{beam,2} \quad \text{(Equation 58)}$$

$$\sum k = k_{col,u} + k_{col,l} + k_{beam,1} + k_{beam,2} \quad \text{(Equation 59)}$$

$$\Delta k = \frac{k_{col}}{\sum k} \quad \text{(Equation 60)}$$

$$\Delta M = |FEM_{beam1} - FEM_{beam2}| \quad \text{(Equation 61)}$$

$$M_{Ed} = \Delta k \times \Delta M \quad \text{(Equation 62)}$$

Where:
- k_{col} is the stiffness of the column in question.
- $k_{beam,1}$, $k_{beam,2}$ is the stiffness of beams 1 and 2, respectively, where beams 1 and 2 are on either side of the column in the frame.
- I is the area moment of inertia, as per *Section 3.6.4.1*.
- L is the length of beam 1 or 2, respectively.
- $k_{col,u}$, $k_{col,l}$ is the stiffness of the upper column and lower column respectively. For the topmost floor, $k_{col,u}$ was taken as 0 as there was no column above it.
- Δk is the effective stiffness.
- ΔM is the difference between the fixed end moments of the beams.
- M_{Ed} is the bending moment acting on the column under consideration.

3.6.6. Minimum and Maximum Reinforcement

The minimum and maximum quantities of reinforcement that are allowable in a column section are described in *Clause 9.5.1 of EN1992*. The provided reinforcement area should be larger than the value calculated from Equation 63, and less than 4% of the column's cross-sectional area, which is the maximum reinforcement area allowed per the code (The European Union, 2004).

$$A_{s,min} = \frac{0.1N_{Ed}}{0.87f_{yk}} \; or \; 0.002A_c, whichever\ is\ greater \quad \text{(Equation 63)}$$

3.7. Code of Practice CP114: Column Design Methodology

3.7.1. Material Properties

The concrete and steel reinforcement material properties established in the simply supported beam design were also applicable to the following column design. A summary of these values is shown in Table 12, and reference to the corresponding sections is required for further information. The permissible stresses in the concrete and the reinforcement are also presented in Table 12, with the relevant section reference.

Table 12: Concrete and steel reinforcement material properties as per CP114

Material Property	Symbol	Section Reference
Concrete cube compressive strength	f_{ck}	3.4.1
Mild reinforcement yield strength	f_y	3.4.2
Hot-rolled deformed reinforcement yield strength	f_y	3.4.2
Concrete density	ρ_C	3.4.3
Permissible Stresses in the Concrete	**Symbol**	**Section Reference**
Direct compressive strength	ρ_{oc}	3.4.10
Compressive strength due to bending	ρ_{ob}	3.4.10
Permissible Stresses in the Reinforcement	**Symbol**	**Section Reference**
Tensile stress in mild steel bars (Φ ≤40mm)	ρ_{st}	3.4.10
Compressive stress in mild steel bars (Φ ≤40mm)	ρ_{sc}	3.4.10
Tensile stress in deformed, high-yield bars (Φ ≤20mm)	ρ_{st}	3.4.10
Compressive stress in deformed, high-yield bars (Φ ≤20mm)	ρ_{sc}	3.4.10

3.7.2. Loading

The loads applied to the column are broken into dead loads and live loads. The dead loads refer to the loads which are permanently applied to the element, including the self-weight of the beams and slab supported by the column. The live loads refer to the loads that are temporarily applied to the element, such as general office loads and wind loads.

3.7.2.1. Dead Loads

The dead loads supported by the column are the self-weight of the reinforced concrete beams, and the reinforced concrete slab resting thereon. The self-weights of the beam and slab are determined by Equations 64 and 65, respectively. Therefore, the dead load is the sum of the self-weights of the beam and the supported slab.

$$DL(beam) = \left(\frac{l_1+l_2}{2}\right) \times h \times \rho_c \qquad \text{(Equation 64)}$$

$$DL(slab) = \left(\frac{l_1+l_2}{2}\right) \times d_{slab} \times \rho_c \qquad \text{(Equation 65)}$$

Where:

- $DL(beam)$ is the self-weight of the beam, calculated as a force per metre length of the beams in the frame.
- $DL(slab)$ is the self-weight of the slab, calculated as a force per metre length of the beams in the frame.

- l_1 is the beam span on the one side of the column.
- l_2 is the beam span on the other side of the column.
- h is the beam height.
- ρ_c is the concrete density, as per *Section 3.7.1*.

3.7.2.2. Live Loads

The *British Standard B.S.9399 Part 1: Loading for Buildings* was the code used for determining the live loading in a building.

3.7.2.3. Load Factor Method

The load factor of 1.8 is used in CP114, and it is defined as per Equation 26, which is the safety factor that is applied to the design loads.

$$Load\ Factor\ LF = \frac{Ultimate\ strength\ of\ the\ beam}{Working\ load} = 1.8 \qquad \text{(Equation 26)}$$

3.7.2.4. Loading Summary

The overall loading according to CP114 is the sum of the dead loads and the live loads, as shown in Equation 29, which is then multiplied by the Load Factor, as shown in Equation 30.

$$P = DL + LL \qquad \text{(Equation 29)}$$

$$P_{LF} = 1.8 \times P \qquad \text{(Equation 30)}$$

3.7.3. Short versus Long Columns

In CP114, columns are classified as either "short" or "long". The key differences between the two are similar to that of the short and slender columns from the Eurocodes, as discussed in *Section 3.6.4*. Therefore, the slenderness ratio of a column is checked according to the steps outlined below.

Columns are treated as short columns when the slenderness ratio is less than 15 (Codes of Practice Committee for Building , 1969). The slenderness ratio is taken as the ratio of the effective length of the column to the smallest lateral dimension of the cross-section of the column, as per Equation 66. The effective column length is determined from *Table 19 in CP114*. If the column's slenderness limit falls between 15 and 57, then the column is treated as a long column and is subjected to second-order effects. The slenderness ratio of the column should not exceed 57. A summary of the slenderness limits is shown in Equations 67 and 68.

$$\lambda = \frac{l_0}{b} \qquad \text{(Equation 66)}$$

$$if\ \lambda < 15, column\ is\ short \qquad \text{(Equation 67)}$$

$$if\ 15 \geq\ \lambda < 57, column\ is\ long \qquad \text{(Equation 68)}$$

Where:

- λ is the slenderness ratio.
- l_0 is the effective length of the column.
- b is the smallest lateral dimension of the cross-section of the column.

3.7.4. Axial Load

The axial load applied to the column is the product of the factored load P_{LF} from Equation 30 multiplied by half the length of each of the beams that it supports in the frame.

The critical axial load on the column is the sum of the loads of all the floors above it. Therefore, the ground floor column sustains the maximum axial load from all the floors above it.

3.7.5. Required Reinforcement

The area of the reinforcement required in the elements cross section is shown in Equation 69, (Codes of Practice Committee for Building , 1969). This is given as a minimum and maximum percentage of reinforcement.

$0.8\%A_c > A_{s,req} < 8\%A_c$ (Equation 69)

Where:
- $A_{s,req}$ is the area of cross-sectional reinforcement required in the column.
- A_c is the cross-sectional area of the concrete column.

The reinforcement provided in the section should be more than or equal to the required reinforcement calculated in Equation 69. The number of reinforcement bars and their cross-sectional area are determined using Table 7.

3.7.6. Design for Short Columns Subject to Direct Load and Bending

The column cross-section is assumed to be controlled by compression when the axial load from *Section 3.7.4* exceeds the limiting load P_b, as per Equations 70 and 71.

$P_b = \rho_{oc} b d_1 X - A_{sc}(\rho_{st} - \rho_{sc})$ (Equation 70)

$If\ P > P_b, the\ section\ is\ controlled\ by\ compression$ (Equation 71)

Where:
- P_b is the limiting load.
- P is the axial load determined in *Section 3.7.4*.
- ρ_{oc} is the permissible stress of the concrete in direct compression, as per *Section 3.7.1*.
- d_1 is the effective depth of the tensile reinforcement.
- A_{sc} is the area of the compressive reinforcement, which for the conditions of bending is equal to half of the total area of reinforcement in the column.
- ρ_{st} is the permissible tensile stress in the reinforcement, as per *Section 3.7.1*.
- ρ_{sc} is the permissible compressive stress in the reinforcement, as per *Section 3.7.1*
- X is shown in Equations 72 and 73, respectively for mild steel and high-yield steel, where E_s is the secant modulus of elasticity of steel at a stress of $1.8\rho_{st}$.

$X = \frac{585}{690+1.8\rho_{st}}$ (Equation 72)

$X = \frac{585}{690+\frac{375\rho_{st}\times10^3}{E_s}}$ (Equation 73)

If the section is determined to be controlled by compression, then the permissible load P on the column is related to the permissible load P_o for an axially loaded column. The axial load P_o which is permissible on a short, reinforced column should not exceed the value given in Equation 74.

$$P_o = \rho_{oc} A_c + \rho_{sc} A_{sc} \qquad \text{(Equation 74)}$$

Where:

- P_o is the permissible axial load.
- ρ_{sc} is the permissible compressive stress in the reinforcement, as per *Section 3.7.1*
- A_c is the cross-sectional area of concrete, minus the reinforcement area.
- ρ_{sc} is the permissible compressive stress in the reinforcement, as per *Section 3.7.1*
- A_{sc} is the cross-sectional area of the longitudinal steel.

3.8. Embodied Carbon Calculation Methodology

To assist designers in understanding the embodied carbon of various products and materials, the Building Services Research and Information Association (BSRIA) developed an embodied carbon guide, entitled “The Inventory of Carbon and Energy”, commonly known as ICE. This was created to assist the United Kingdom (UK) in meeting its carbon emissions reduction objectives of at least 80% by 2050 (Hammond & Jones, 2011).

ICE is a materials database of the embodied carbon of different products and materials used within the construction sector. As part of ICE, the *Embodied Carbon Calculator for Concrete* software was developed (Circular Ecology, 2019). This software was used to calculate the embodies carbon of the reinforced concrete elements designed in this research project. This software takes multiple factors into account, such as the volume of concrete and reinforcement in the element, as well as the concrete mix design.

This study was used to determine the carbon dioxide savings purely from the optimization of the element dimensions and the reductions in material quantities, without considering the quality and composition of those materials. Therefore, SCMs were not introduced into the scope of this research.

To establish the embodied carbon changes based solely on the optimized material quantities, it was assumed that the concrete mix designs from both the current and historic design codes were the same. This was done to ensure that the embodied carbon per given unit volume of concrete was the same.

This is a broad assumption which has its limitations. For example, the type of cement used could differ between the decades. The required water content and cement content might differ. The composition of the concrete might differ in terms of aggregate sizes, quality, and quantity. However, these factors have not been considered in this research, and therefore it was assumed that a hundred percent CEM I cement was used in the concrete mix design.

As part of this research project, the Embodied Carbon Calculator created by Circular Ecology was used to determine the embodied carbon of the designed elements. The boundaries of the calculator are normally set as cradle to site. However, the transport distance was set as zero, which allowed for a cradle to gate investigation.

To calculate the embodied carbon for each of the simply supported beam and column designs discussed in this section, the following inputs were required.

3.8.1. Concrete Mixture Inputs

The following information about the concrete mix is required: whether the concrete is precast or placed in situ, the cement type, the cement quantity, the water-to-cement ratio, the water content, and the aggregate quantity.

Different reinforced concrete sections were generated from each of the codes due to the differences in the analysis procedures specified in each of them. Therefore, using a similar embodied carbon for each of the concrete design mixes, other than any explicit concrete mix design requirements stipulated by the code. any changes in the carbon dioxide emissions were derived directly from the optimization of the dimensions of the section of the member, and the required reinforcement to sustain the load.

3.8.2. Steel Reinforcement Inputs

The volume of the steel reinforcement used, as well as the type of steel used, in each of the members is required as an input for the calculator.

3.9. Summary

This chapter described the manual design calculations undertaken to determine the embodied carbon of a simply supported beam and column element, using the Eurocodes and the Code of Practice CP114. A cradle to gate LCA boundary was considered.

The approach consisted of two parts; part one was the design of a simply supported beam according to the Eurocodes and then to Code of Practice CP114. Part two was the design of a column according to the Eurocodes and then to Code of Practice CP114. The results are presented in Chapter 4.

4. RESULTS

4.1. Introduction

The results of the manual design calculations carried out to determine the embodied carbon of a simply supported beam and column element are presented in this chapter. This chapter shows the detailed analysis of one simply supported beam design and one column design. However, a number of simply supported beams and columns were designed, whilst changing different variables such as the lengths, concrete strengths, or the design loads. A summarised set of results for these additional iterations are described and presented in Chapter 5.

4.2. Simply Supported Beam Design

A simply supported reinforced concrete beam, shown in Figure 8, was designed using two different structural design codes; the current Eurocodes and the withdrawn Code of Practice CP114. *Section 4.3* presents the design considerations and parameters which were kept constant across both designs. *Section 4.4* presents the calculations and results according to the Eurocodes, while *Section 4.5* presents the calculations and results according to CP114. For a summary of all simply supported beam results refer to *Section 4.6*.

4.3. Design Scenario and Constants

The factors that were kept constant throughout both the Eurocode design and the CP114 design are noted below.

- The beam is simply supported, with pinned end supports, situated indoors in an office building in Dublin, Ireland. Refer to Figure 8 for a depiction of the beam.
- The beam is singly reinforced and therefore only requires tension reinforcement. This will allow for an accurate comparison between the beam designed to the Eurocodes and the CP114 code.
- The simply supported beam has a length of 4m. In industry, spans typically range from 3 m to 6 m. Therefore, a standard span of 4 m was chosen.
- The beam has a breadth of 150 mm.
- The beam is constructed from reinforced concrete.
- The beam supports regular office loads, its self-weight, and the weight of the 150mm deep slab above it.
- For the initial iteration of the beam design, a depth of 600 mm was considered.

An iterative process was used to determine what is the optimal height of the beam. The beams height was gradually decreased in the calculations, which were carried out in an excel spreadsheet, to determine at which height it would fail, either in shear, bending, or deflection. This is explained further in Section 4.4.12.

4.4. Eurocodes: Simply Supported Beam Results

4.4.1. Design Life, Exposure Class, and Concrete Strength Class

4.4.1.1. Design Life

For “building structures and other common structures” a design working life of 50 years was selected, as shown in Table 4, (The European Union, 2005).

4.4.1.2. Exposure Class

Dublin, Ireland is cool and humid throughout the year with an average relative humidity of 73% and can be classified as a moderately humid environment (Climates to travel, 2020). The exposure class of a beam set inside a building in Dublin, Ireland, was determined to be XC3, per *Section 3.3.1.2*, in Table 5.

- Exposure Class XC3: Moderate humidity. This exposure class refers to elements inside buildings with moderate or high air humidity, where corrosion is induced by carbonation.

4.4.1.3. Concrete Strength Class

An indicative minimum strength class of C30/37 was required for an XC3 exposure class with a design life of 50 years, as per *Section 3.3.1.3*. This corresponds to a cylinder strength of 30MPa at 28 days, and an equivalent cube strength of 37 MPa, as summarised below.

- Characteristic cylinder strength f_{ck} = 30 MPa
- Characteristic cube strength $f_{ck,cube}$ = 37 MPa

4.4.2. Reinforcement Tensile Strength

The grade of reinforcement used in this design was B500B (Heaton Manufacturing, 2020). This indicates that the bars were dented or ribbed along their length, giving the reinforcement more grip in the concrete. From *Annex C of EN1992*, Class B500B reinforcement has a characteristic yield strength of 500MPa, as summarised below.

- Characteristic strength of high yield steel f_{yk} = 500 MPa.

4.4.3. Design Strength of Materials

The material partial safety factors are provided below, as per *Section 3.3.3*.

- Concrete partial factor γ_C = 1.5
- Steel partial factor γ_S = 1.15

For class C30/37 concrete, the design compressive strength is 20 MPa, as in Equation 2. The design strength for the reinforcement is 434.78MPa, as shown in Equation 3.

$$f_{cd} = \alpha_{cc}\frac{f_{ck}}{\gamma_c} = 1.0 \times \frac{30}{1.5} = 20MPa \qquad \text{(Equation 2)}$$

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{500}{1.15} = 434.78MPa \qquad \text{(Equation 3)}$$

Where:
- f_{cd} is the design concrete strength.
- f_{yd} is the design reinforcement strength.

- α_{cc} is the coefficient that accounts for the long-term effects of the load application, recommended as 1.0 in the Eurocodes.
- f_{ck} is the characteristic concrete strength, as per *Section 4.4.1*.
- f_{yk} is the characteristic reinforcement strength, as per *Section 4.4.2*.
- γ_C is the concrete partial factor.
- γ_S is the reinforcement partial factor.

4.4.4. Concrete Density

The concrete density (ρ_C) in the Eurocodes for normal-weight concrete is 24 kN/m³. This value is increased by 1 kN/m³ for a normal percentage of reinforcement. The concrete density ρ_C was therefore taken as 25 kN/m³ for this research project.

4.4.5. Permanent Loads

The self-weight of the beam was determined by Equation 4. An initial, conservative assumption of the depth of the beam was made, as specified in the controlled variables in *Section 4.3*.

$$G_k(beam) = l \times b \times h \times \rho_c = 4 \times 0.15 \times 0.6 \times 25 = 9\ kN \qquad \text{(Equation 4)}$$

Where:
- l is the length of the beam, as per *Section 4.3*.
- b is the breadth of the beam, as per *Section 4.3.*
- h is the initial estimated beam height, as per *Section 4.3.*
- ρ_c is the concrete density, as determined in *Section 4.4.4*.

The weight of the slab which is supported by the beam is a permanent load and was calculated by Equation 5. The tributary surface area of the supported slab supported by the beam under consideration is shown in Figure 16. The beam under consideration supports the self-weight of half of the slab on either side of it. The depth of the regularly reinforced one-way spanning concrete slab was assumed to be150mm. Any weights from finishes were ignored.

$$G_k(slab) = d_{slab} \times b_{supp} \times l_{supp} \times \rho_c = 4 \times 4 \times 0.15 \times 25 = 60\ kN \qquad \text{(Equation 5)}$$

Where:
- d_{slab} is the depth of the supported slab, as per *Section 4.3.*
- l_{supp} is the length of the slab supported by the beam.
- b_{supp} is the breadth of the slab supported by the beam.
- ρ_c is the concrete density, as determined in *Section 4.4.4*.

Therefore, the sum of the permanent loads G_k supported by the beam was 69 kN.

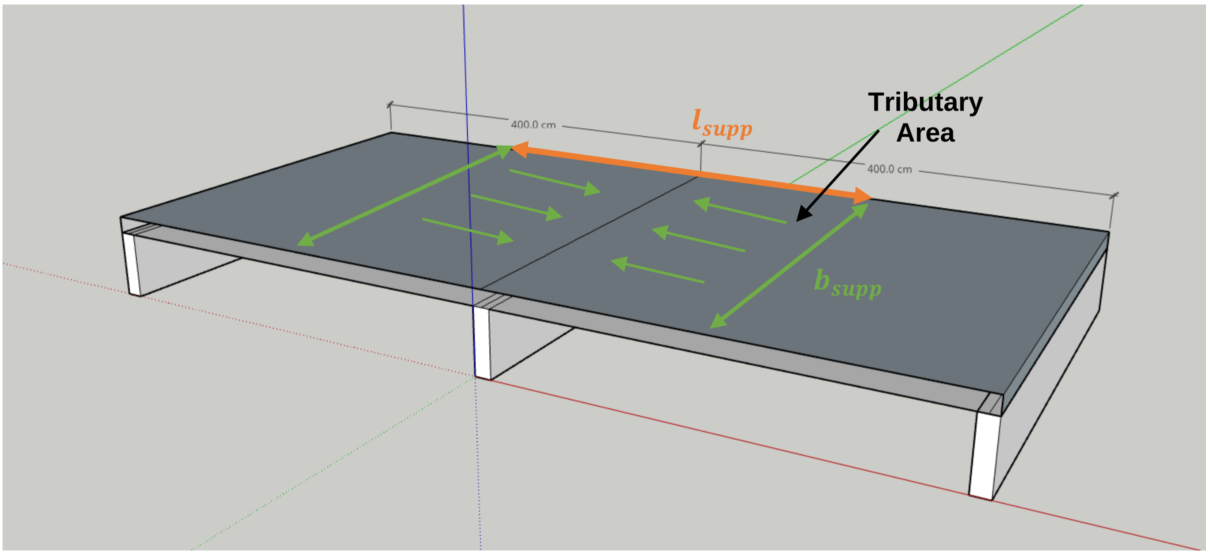

Figure 16: 3D image of analysed 4m RC beam (middle beam) supporting a 150mm one-way spanning slab

4.4.6. Imposed Loads

The simply supported beam under consideration was located in an office building. Category B: Office Areas was chosen as the usage category for this design scenario. The corresponding imposed loads for office areas have been summarised in Table 13.

Table 13: Imposed Loads as per EN1991-1-1 (The European Union, 2002)

Category B: Office Areas Imposed Loads	
$q_{k,min}$	2 kN/m²
$q_{k,max}$	3 kN/m²
$Q_{k,min}$	1.5 kN
$Q_{k,max}$	4.5 kN

The wind load ($Q_{k,2}$) design methodology and calculations were not detailed here as the beam is an interior element and therefore would not be subjected to significant wind loads.

4.4.7. Loading Summary

The resultant loads which act on the beam are summarised as follows:

- The permanent load acting on the beam: G_k = 69 kN.
- The imposed load from general office activity: An average between the minimum and maximum values in Table 13 was used: $Q_{k,1} = \frac{1.5+4.5}{2} = 3\ kN$. Other loads such as furniture and storage were ignored.

4.4.8. Design Loads, Partial Factors, and Combinations of Actions

The combination values for this design scenario are shown in Table 14. These apply to the primary imposed loads ($Q_{k,1}$) and the secondary imposed wind loads ($Q_{k,2}$). However, in this design scenario for an interior beam, the imposed wind loads were assumed to be zero, and therefore do not apply.

Table 14: Partial factors for combination actions (The European Union, 2002)

Imposed Action	Symbol	Value	Reference
Category B: office areas	$\Psi_{0,1}$	0.7	Table A1.1 EN1990
Wind loads on buildings	$\Psi_{0,2}$	0.6	

The overall loading on the beam was taken as the maximum load from the iteration of equations summarised in Table 15, based on Equation 6, where E_d represents the design load applied to the element, with $E_{d,max}$ as the maximum (and therefore most unfavourable) design loading.

$$E_d = \sum_{j\geq 1} \gamma_{G,j} G_{k,j} + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \qquad \text{(Equation 6)}$$

Table 15: Overall factored beam loads

Combination	Equation	Value
Combination 1: UN-F	$E_d = 1.35G_k + 1.5Q_{k,1}$	97.65 kN
Combination 1: F	$E_d = G_k$	69.00 kN
Combination 2: UN-F	$E_d = 1.35G_k + 1.5Q_{k,2}$	93.15 kN
Combination 2: F	$E_d = G_k + 1.5Q_{k,2}$	69.00 kN
Combination 3: UN-F	$E_d = 1.35G_k + 1.5Q_{k,1} + \Psi_{0,2} \times 1.5Q_{k,2}$	97.65 kN
Combination 3: F	$E_d = G_k + \Psi_{0,2} \times 1.5Q_{k,2}$	69.00 kN
Combination 4: UN-F	$E_d = 1.35G_k + \Psi_{0,1} \times 1.5Q_{k,1} + 1.5Q_{k,2}$	96.30 kN
Combination 4: F	$E_d = G_k + 1.5Q_{k,2}$	69.00 kN
SLS	$E_d = G_k + Q_{k,1} + Q_{k,2}$	72.00 kN
	$\boldsymbol{E_{d,max}}$	**97.65 kN**

4.4.9. Reaction Forces and Shear Force Diagram

Figure 9 shows the shear force diagram for the simply supported beam under consideration, with reaction forces calculated as per Equation 7.

$$Reaction\ A = Reaction\ B = \frac{E_{d,max}}{2} = 48.83\ kN \qquad \text{(Equation 7)}$$

4.4.10. Bending Moments and Bending Moment Diagram

Figure 10 shows the bending moment diagram for the simply supported beam under consideration. The maximum moment of the beam was determined as per Equation 8. This results in the maximum moment M_{max} of 97.65 kNm at the centre of the beam.

$$M_{max} = \frac{E_{d,max} l}{4} = 97.65kNm \qquad \text{(Equation 8)}$$

Where:
- $E_{d,max}$ is the maximum design load, as determined in *Section 4.4.8*.
- l is the length of the beam.

4.4.11. Concrete Cover

The nominal concrete cover (c_{nom}) was calculated as 35 mm from Equations 9 and 10.

$$c_{nom} = c_{min} + \Delta c_{dev} = 35mm \qquad \text{(Eq. 9)}$$

$$c_{min} = max\{c_{min,b}; c_{min,dur} + \Delta c_{dur,\gamma} - \Delta c_{dur,st} - \Delta c_{dur,add}; 10mm\} = 25mm \quad \text{(Eq. 10)}$$

Where:

- c_{min} is the minimum concrete cover that should be provided.
- Δc_{dev} is the allowance for deviation and is recommended as 10mm, as per *EN1992*.
- $c_{min,b}$ is the minimum cover due to the bond requirements and is taken as the diameter of the separate reinforcement bars. The assumed diameter of the steel reinforcing bar was 12mm, as this is a diameter that is commonly used in practice.
- $c_{min,dur}$ is the minimum cover due to environmental conditions which depends on the exposure class. As per *EN1992*, the Structural Class for an element or structure with a design working life of 50 years is S4. This corresponds to a value of 25 mm for an exposure class of XC3 (The European Union, 2004).
- $\Delta c_{dur,\gamma}$ is the additive safety element, recommended as 0mm in *EN1992*.
- $\Delta c_{dur,st}$ is the possible reduction in cover due to the use of stainless steel. The recommended value is 0 mm.
- $\Delta c_{dur,add}$ is the possible reduction in cover due to additional protection. The recommended value is 0 mm.

4.4.12. Deflection and Minimum Depth

As per *EN1992*, the basic span over effective depth ratio is 20 for a simply supported beam with lightly stressed concrete, per Equation 11. Using this ratio, the limiting effective depth of the beam was determined to be 200 mm (The European Union, 2004).

$$\frac{l}{d} = \frac{4000}{d} \leq 20 \;\; \therefore d = 200\ mm \quad \text{(Equation 11)}$$

The overall limiting height of the beam was determined using Equation 12. The diameter of the reinforcement ($\emptyset_{rebar}$) was assumed to be 12 mm, and the concrete cover was taken as the nominal cover of 35 mm as calculated in *Section 4.4.11*. Therefore, the minimum height of the beam was 241 mm, to avoid significant deflections.

$$h_{limit} = d + c_{nom} + \frac{\emptyset_{rebar}}{2} = 200 + 35 + \frac{12}{2} = 241\ mm \quad \text{(Equation 12)}$$

With a limit of 241mm for the height, it was determined that 420mm is the minimum and optimal height of the beam under consideration. This was determined by iteratively changing the height of the beam to establish at which height it would fail. At 420mm, the beam could still withstand the load and complied with all the checks required for beam design, such as:

- The beam stays within the limiting span/depth ratio, as per Equation 11, as well as the in-depth deflection check described in Section 4.4.21.
- The beam is singly reinforced, as per Section 4.4.13.
- The neutral axis falls within the required x/d ratio, as per Section 4.4.18.
- The beam will not fail in shear, as per section 4.4.19.
- The beam will not fail in bending, as the moment which can be resisted by the beam is greater than the maximum moment applied to the beam, as per Section 4.4.20.

Working backward from Equation 12, an effective depth of 379 mm was determined, as shown in Equation 75 below.

$$d_{effective} = h - c_{nom} - \frac{\emptyset_{rebar}}{2} = 420 - 35 - \frac{12}{2} = 379\ mm \qquad \text{(Equation 75)}$$

4.4.13. Singly Reinforced Section

The limiting factor K was determined according to Equation 13 as less than 0.167. Therefore, the section only required tensile steel, and no compression steel at the top portion of the cross-section needed to be considered.

$$K = \frac{M_{max}}{bd^2 f_{ck}} = 0.151 \leq 0.167 \qquad \text{(Equation 13)}$$

Where:

- M_{max} is the maximum bending moment, as determined in *Section 4.4.10.*
- b is the breadth of the beam.
- d is the effective depth of the tensile reinforcement, as determined in *Section 4.4.12*.
- f_{ck} is the characteristic concrete strength, as determined in *Section 4.4.1*.

4.4.14. Lever Arm

The lever arm was calculated from Equation 14 as 319 mm. This is less than 95% of the effective depth, which complies with Equation 15.

$$z = d\left[0.5 + \sqrt{0.25 - \frac{K}{1.134}}\right] = 319\ mm \qquad \text{(Equation 14)}$$

$$z = 319\ mm < 0.95d = 0.95(379) = 360\ mm \qquad \text{(Equation 15)}$$

Where:

- z is the lever arm.
- d is the effective depth of the tensile reinforcement, as determined in *Section 4.4.12*.
- K is the limiting factor determined in *Section 4.4.13*.

4.4.15. Required Reinforcement

The required area of reinforcement in the cross section was calculated using Equation 16 as 600 mm².

$$A_{s,req} = \frac{M_{max}}{0.87 f_{yk} z} = 704\ mm^2 \qquad \text{(Equation 16)}$$

Where:

- M_{max} is the maximum bending moment, as determined in *Section 4.4.10*.
- f_{yk} is the characteristic strength of the reinforcement, as per *Section 4.4.2*.
- z is the cross-sectional lever arm, as determined in *Section 4.4.14*.

4.4.16. Provided Reinforcement

From Table 16, seven reinforcement bars each with a diameter of 12 mm were determined to be sufficient to maintain the section, as summarised below in Equation 76.

$A_{s,prov} = 791\ mm^2 > 704\ mm^2$ (Equation 76)

Table 16: Reinforcement area provided for the simply supported beam designed to the Eurocodes (StructX, 2022)

	Cross-sectional Area (mm²)								
	Bar Diameter (mm)								
No. of Bars	**10**	**12**	**16**	**20**	**24**	**28**	**32**	**36**	**40**
1	78	113	201	314	452	616	804	1020	1260
2	156	226	402	628	904	1232	1608	2040	2520
3	234	339	603	942	1356	1848	2412	3060	3780
4	312	452	804	1256	1808	2464	3216	4080	5040
5	390	565	1005	1570	2260	3080	4020	5100	6300
6	468	678	1206	1884	2712	3696	4824	6120	7560
7	546	791	1407	2198	3164	4312	5628	7140	8820
8	624	904	1608	2512	3616	4928	6432	8160	10080
9	702	1017	1809	2826	4068	5544	7236	9180	11340
10	780	1130	2010	3140	4520	6160	8040	10200	12600
Min. Hole (mm)	12	15	20	25	29	34	39	44	49

4.4.17. Minimum and Maximum Reinforcement

The reinforcement provided ($A_{s,prov}$) in the section is more than the minimum required reinforcement area ($A_{s,min}$) 74 mm², as shown in Equation 17, and therefore sufficient reinforcement was provided. The provided reinforcement ($A_{s,prov}$) is also less than the maximum allowable reinforcement area ($A_{s,max}$) of 2274 mm², as shown in Equation 18. The section is therefore not over-reinforced.

$A_{s,prov} > A_{s,min} = 0.13\% bd = 74\ mm^2$ (Equation 17)

$A_{s,prov} < A_{s,max} = 4\% bd = 2274\ mm^2$ (Equation 18)

Where:
- $A_{s,prov}$ is the provided area of reinforcement, as determined in *Section 4.4.16*.
- $A_{s,min}$ is the minimum area of reinforcement required to maintain the section.
- $A_{s,max}$ is the maximum area of reinforcement which should be provided in the section.
- b is the breadth of the beam.
- d is the effective depth of the tensile reinforcement, as determined in *Section 4.4.12*.

4.4.18. Cross-Sectional Neutral Axis

The neutral axis (x) for the simply supported beams cross-section is shown in Figure 11. Its position was determined using Equations 19 and 20. The neutral axis was determined to be 143mm from the top surface of the cross-section.

$F_c = F_s$ (Equation 19)

$\eta f_{cd} b \lambda x = f_{yd} A_s \ \therefore x = 143\ mm$ (Equation 20)

Where:
- F_c represents the compressive forces in the concrete.
- F_s represents the tensile forces in the steel reinforcement.

- η defines the effective strength of the section and has a value of 1.0 for concrete strength classes less than 50 MPa.
- f_{cd} is the design value of the concrete compressive strength, as per *Section 4.4.3*.
- b is the breadth of the beam cross-section.
- λ defines the effective height of the compressive zone of the cross-section and has a value of 0.8 for concrete strength classes less than 50MPa.
- x is the depth to the neutral axis of the cross-section.
- f_{yd} is the design tensile strength of the reinforcement, as per *Section 4.4.3*.
- A_s is the provided area of tension reinforcement, as per *Section 4.4.16*.

The limiting x/d ratio is defined in Equation 21. This was determined to be 0.38, as shown in Equation 77, which is less than 0.617 and the steel will therefore yield as required for a singly reinforced section.

$$\frac{x}{d} < \frac{1}{\frac{f_{yk}}{\gamma_S E_S \varepsilon_{cu3}}+1} = 0.617 \qquad \text{(Equation 21)}$$

$$\frac{x}{d} = \frac{143}{379} = 0.38 \leq 0.617 \qquad \text{(Equation 77)}$$

Where:
- x is the depth to the neutral axis, as determined above.
- d is the effective depth of the tension reinforcement, as per *Section 4.4.12*.
- f_{yk} is the characteristic yield strength of the reinforcing steel, as per *Section 4.4.2*.
- γ_s is the partial factor for steel, as per *Section 4.4.3*.
- E_s is the design value of the modulus of elasticity of reinforcing steel, taken as 200GPa.
- ε_{cu3} is the shortening concrete strain, and has a value of 0.0035, as per *Section 3.3.13*.

4.4.19. Shear Resistance

The maximum shear resistance of the beam is larger than the design shear forces, as shown in Equations 22 and 78. Therefore, the element will not fail in shear.

$$V_{Rd,max} = 0.124bd\left(1 - \frac{f_{ck}}{250}\right)f_{ck} = 186\ kN \qquad \text{(Equation 22)}$$

$$V_{Rd,max} = 186\ kN \geq V_{Ed,max} = 97.7\ kN \qquad \text{(Equation 78)}$$

Where:
- b is the breadth of the beam.
- d is the effective depth of the tensile reinforcement, as per *Section 4.4.12*.
- f_{ck} is the characteristic strength of the concrete, as per *Section 4.4.1*.

4.4.20. Bending Moment of Resistance

The maximum moment that the cross-section can resist is larger than the maximum applied bending moment. This is shown in Equations 23 and 79. Therefore, the beam will not fail in bending.

$$M_r = A_s f_{yd} z = 109.7\ kNm \qquad \text{(Equation 23)}$$

$M_r = 109.7\ kNm \geq M_{max} = 97.7\ kNm$ (Equation 79)

Where:

- M_r is the maximum moment which the cross-section can resist.
- A_s is the cross-sectional area of reinforcement provided, as per *Section 4.4.16*.
- f_{yd} is the design tensile strength of the reinforcement, as per *Section 4.4.3*.
- z is the cross-section's lever arm, as determined in *Section 4.4.14*.
- M_{max} is the maximum applied bending moment, as determined in *Section 4.4.10*.

4.4.21. Deflection Check

Deflections do not need to be checked explicitly if they conform to the limiting span/depth ratios in Equations 24 or 25.

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}} + 3.2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right]\ if\ \rho \leq \rho_0 \qquad \text{(Equation 24)}$$

$$\frac{l}{d} = K\left[11 + 1.5\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho - \rho'}\right) + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right]\ if\ \rho > \rho_0 \qquad \text{(Equation 25)}$$

Where:

- $\frac{l}{d}$ is the limiting span/depth ratio. l is the length of the beam, and d is the effective depth of the beam, as determined in *Section 4.4.12*.
- K is the factor that takes different structural systems into account and is equal to 1.0 for a simply supported beam, as per *Table 7.4N in EN1992*.
- f_{ck} is the characteristic strength of the concrete, as per *Section 4.4.1*.
- ρ_0 is the reference reinforcement ratio and is equal to $10^{-3}\sqrt{f_{ck}}$ as per *EN1992*. Therefore, for a characteristic concrete strength of 30MPa, $\rho_0 = 0.00548$.
- ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads. Therefore, $\rho = \frac{A_{s,req}}{bd} = \frac{704}{150 \times 379} = 0.012$.
- ρ' is the required compression reinforcement ratio at midspan to resist the moment due to the design loads. This was taken as zero, as there was no compression reinforcement provided.

From the above, it is evident that $\rho > \rho_0$, and therefore Equation 25 applies to this element, which results in a limiting $\frac{l}{d}$ value of 14.6. The code specifies that a modification factor should be applied to beams with spans larger than 7 m, which did not apply to this design case, and was therefore not used. In Equation 80, the actual span/depth ratio was calculated to be 10.6, which is less than the limiting value of 14.6, and is therefore sufficient.

$$\left(\frac{l}{d}\right)_{actual} = \frac{4000}{379} = 10.6 < \frac{l}{d} = 14.6 \qquad \text{(Equation 80)}$$

4.5. Code of Practice CP114: Simply Supported Beam Results

4.5.1. Concrete Strength Class

To compare the beam designed from the Eurocodes to the beam designed to the Code of Practice CP114, similar concrete strength classes were considered in the design. In the Eurocode design, it was determined that due to the exposure class of the beam, it should be constructed using concrete with a characteristic cylinder strength of 30 MPa, and a characteristic cube strength of 37 MPa. At the time when CP114 was in use, the cylinder strength tests were not yet incorporated into testing methods and designs. Therefore, the cube strength of 37 MPa was considered.

From Table 8, it was evident that the nominal concrete design mixture with a compressive strength closest to that of the 37 MPa is the 1:1:2 concrete design mixture. This directly translates into one part cement to one-part fine aggregate (sand) to two parts coarse aggregate. It has a compressive strength at 28 days of 30 MPa, as per the works test, as summarised below. It is important to note that this strength is lower than that used in the Eurocode design.

- Cube compressive strength at 28 days f_{ck}= 30 MPa

4.5.2. Reinforcement Tensile Strength

For this research, the reinforcement specified in *B.S.4449-1969* was used. A summary of the reinforcement yield strengths is shown below.

- Mild rebar yield strength f_y= 250 MPa
- Hot rolled deformed rebar yield strength f_y = 410 MPa

4.5.3. Concrete Density

As per *Section 3.4.3*, the Concrete Density ρ_c is 24 kN/m³.

4.5.4. Loading

4.5.4.1. Dead Loads

The self-weight of the beam was calculated using Equation 27. The self-weight of the regularly reinforced supported slab was calculated using Equation 28. Therefore, the sum of the dead loads supported by the beam is 66.96 kN.

$$DL(beam) = l \times b \times h \times \rho_c = 4 \times 0.15 \times 0.65 \times 24 = 9.36\ kN \qquad \text{(Eq. 27)}$$

$$DL(slab) = d_{slab} \times b_{supp} \times l_{supp} \times \rho_c = 4 \times 4 \times 0.15 \times 24 = 57.60\ kN \qquad \text{(Eq. 28)}$$

Where:

- $DL(beam\ or\ slab)$ is the dead load of the beam or the supported slab, respectively.
- ρ_c is the concrete density, as per *Section 4.5.3*.
- l is the length of the beam, which is a controlled variable as per *Section 4.3*.
- b is the breadth of the beam, which is a controlled variable as per *Section 4.3*.
- h is the height of the beam, taken as 650mm as the initial estimate.
- d_{slab} is the depth of the supported slab, a controlled variable as per *Section* 4.3.

- b_{supp} is the breadth of the slab supported by the beam, as determined in *Section 3.4.5* and shown in Figure 16.
- l_{supp} is the length of the slab supported by the beam, as determined in *Section 3.4.5* and shown in Figure 16.

4.5.4.2. Live Loads

The live loads acting on the beam are the imposed floor loads from an office scenario. A summary of the live loads for "Category B: offices for general use" is tabulated in Table 17.

Table 17: Imposed loads as per B.S.9399

Category B imposed loads: offices for general use	
$LL_{distributed\ loads}$	2.5 kN/m²
$LL_{concentrated\ load}$	2.7 kN

4.5.4.3. Loading Summary

The overall loading according to CP114 is the sum of the dead loads and the live loads, as shown in Equation 29. By taking the load factor method into account, the overall load P was multiplied by the load factor, as shown in Equation 30.

$$P = DL + LL = 69.66\ kN \qquad \text{(Equation 29)}$$

$$P_{LF} = 1.8 \times P = 125.39\ kN \qquad \text{(Equation 30)}$$

4.5.5. Reaction forces

The reaction forces were calculated in the same manner as *Section 4.4.9*, as shown in Equation 31, where P_{LF} is the factored design load as determined in *Section 4.5.4*.

$$Reaction\ A = Reaction\ B = \frac{P_{LF}}{2} = 62.70\ kN \qquad \text{(Equation 31)}$$

4.5.6. Bending Moment

The maximum bending moment was calculated in the same manner as *Section 4.4.10*, as shown in Equation 32.

$$M_{max} = \frac{P_{LF}l}{4} = 125.39\ kNm \qquad \text{(Equation 32)}$$

Where:
- M_{max} is the maximum bending moment.
- P_{LF} is the design load applied to the beam, as per *Section 4.5.4*.
- l is the length of the beam.

4.5.7. Concrete Cover

The diameter of the reinforcing steel was assumed to be 20 mm. As per Section 3.4.8., the cover should not be less than 25mm, nor less than two times the diameter of the reinforcement. Therefore, the required concrete cover was taken as two times the reinforcement diameter. This is 40 mm.

4.5.8. Deflection and Minimum Depth

The permissible span over depth ratio as per *Table 13 of CP114* is 20 for simply supported beams. Therefore, the minimum effective depth to the tension reinforcement was determined to be 200 mm, as per Equation 11. This corresponds to a limiting height of the beam of 250mm, as shown in Equation 12.

$$\frac{l}{d} = \frac{4000}{d} \leq 20 \ \therefore d = 200\ mm \qquad \text{(Equation 11)}$$

$$h_{limit} = d + cover + \frac{\emptyset_{rebar}}{2} = 200 + 40 + \frac{20}{2} = 250\ \ mm \qquad \text{(Equation 12)}$$

Where:

- l is the length of the beam.
- d is the limiting depth of the beam, to maintain the span over depth ratio.
- h_{limit} is the limiting height of the beam, to maintain the span over depth ratio.
- $cover$ is the concrete cover provided, as per *Section 4.5.7*.
- $\emptyset_{rebar}$ is the assumed diameter of the reinforcement, taken as 20 mm.

4.5.9. Permissible Stresses

For a concrete mix design of 1:1:2, the permissible stresses in the concrete and the reinforcement are shown in Table 18, as taken from *CP114 Clauses 303 and 304*.

Table 18: Permissible material stresses for a nominal mix design of 1:1:2, as per CP114

Permissible Stresses in the Concrete		
Direct compressive strength	ρ_{oc}	7.6 MPa
Compressive strength due to bending	ρ_{ob}	10 MPa
Permissible Stresses in Rebar		
Tensile stress in mild steel bars (Φ ≤40mm)	ρ_{st}	140 MPa
Compressive stress in mild steel bars (Φ ≤40mm)	ρ_{sc}	125 MPa
Tensile stress in deformed, high-yield bars (Φ ≤20mm)	ρ_{st}	$0.55f_y = 225.5\ MPa$ & ≤ 230 MPa
Compressive stress in deformed, high-yield bars (Φ ≤20mm)	ρ_{sc}	$0.55f_y = 225.5\ MPa$ & ≤ 175 MPa ∴ 175 MPa

It was determined that no modifications to the permissible stresses in Table 18 applied to this design case. Assuming that the element was loaded after one month, then the Age Factor is taken as 1.0. The beam was determined to have a slenderness ratio of less than the limiting value of 30, as shown in Equation 33. Therefore, no reductions to the permissible stresses were necessary.

$$\frac{l}{b} = \frac{4000}{150} = 26.67 \leq 30 \qquad \text{(Equation 33)}$$

4.5.10. Beam Dimensions

Several iterations of the beam design were used to establish the required depth dimensions of the beam. All the iterations are not described here; however, the optimal required dimensions are shown in Table 19, and were determined based on iterative calculations using the equations from *Sections 3.5.11 to 3.5.13*.

With a limit of 250mm for the height, it was determined that 630mm is the minimum and optimal height of the beam under consideration, regardless of whether mild steel or high-yield steel was used. This was determined by iteratively changing the height of the beam to establish at which height it would fail. At 630mm, the beam could still withstand the load and complied with all the checks required for beam design, such as:

- The beam stays within the permissible span over depth, as per Section 4.5.8.
- The beam stays within the limiting bending ratio, as per Equation 33 in Section 4.5.9.
- The beam will not fail in bending, as the moment which can be resisted by the beam is greater than the maximum moment applied to the beam, as per Section 4.5.12.
- The stresses in the concrete and the steel are less than the permissible stresses in the concrete and the steel prescribed by the CP114 code, as per Section 4.5.13.

Working backward from Equation 12, an effective depth of 580 mm was determined, as shown in Equation 75 below.

$$d_{effective} = h - c_{nom} - \frac{\emptyset_{rebar}}{2} = 630 - 40 - \frac{20}{2} = 580mm \qquad \text{(Equation 75)}$$

Table 19: Optimised beam dimensions as per CP114

Simply supported reinforced concrete beam dimensions using mild steel bars		
Length	l	4000 mm
Width	b	150 mm
Height	h	630 mm
Effective Depth	d_1	580 mm
Simply supported reinforced concrete beam dimensions using high-yield steel bars		
Length	l	4000 mm
Width	b	150 mm
Height	h	630 mm
Effective Depth	d_1	580 mm

4.5.11. Required Reinforcement

The cross-sectional area of the tensile reinforcement required to ensure that the beam cross-section remains balanced was determined from Equation 34. The required cross-sectional area of mild steel bars versus deformed high-yield steel bars is summarised below.

- Mild steel bars: $A_{st,bal}$ = 2072 mm²
- Deformed high-yield steel bars: $A_{st,bal}$ = 1286 mm²

$$\frac{A_{st,bal}}{bd_1} = \frac{1}{3}\frac{\rho_{ob}}{\rho_{st}} \qquad \text{(Equation 34)}$$

Where:

- $A_{st,bal}$ is the cross-sectional area of the tensile reinforcement required to ensure a balanced cross-section.
- b is the breadth of the element.

- d_1 is the effective depth determined as 580mm, as shown in Table 19.
- ρ_{ob} is the permissible compressive strength due to bending in the concrete, as shown in Table 18.
- ρ_{st} is the permissible tensile stress in the reinforcement, as shown in Table 18.

The provided area of steel was determined from Table 20. The beam with the mild steel reinforcement requires seven 20 mm bars, which provides an area of 2198 mm², and the beam with deformed high-yield steel bars will have five 20 mm bars provided, which provides an area of 1570 mm².

Table 20: Reinforcement areas for the simply supported beam designed to CP114 (PRASAD, n.d.)

	Cross-sectional Area (mm²)								
	Bar Diameter (mm)								
No. of Bars	**10**	**12**	**16**	**20**	**24**	**28**	**32**	**36**	**40**
1	78	113	201	314	452	616	804	1020	1260
2	156	226	402	628	904	1232	1608	2040	2520
3	234	339	603	942	1356	1848	2412	3060	3780
4	312	452	804	1256	1808	2464	3216	4080	5040
5 →	390	565	1005	$\boxed{1570}$	2260	3080	4020	5100	6300
6	468	678	1206	1884	2712	3696	4824	6120	7560
7 →	546	791	1407	$\boxed{2198}$	3164	4312	5628	7140	8820
8	624	904	1608	2512	3616	4928	6432	8160	10080
9	702	1017	1809	2826	4068	5544	7236	9180	11340
10	780	1130	2010	3140	4520	6160	8040	10200	12600
Min. Hole (mm)	12	15	20	25	29	34	39	44	49

4.5.12. Resistance Moments

The moment resisted by the beam depends on Equations 35 and 36. The maximum resistance moment that can be sustained by the beam is the lesser of the two answers from these equations.

The area of reinforcement provided was 2198 mm² and 1570 mm², respectively, for a beam with mild steel bars and high-yield steel bars, as per *Section 4.5.11*. The lever arm was therefore determined to be 426.14 mm and 402.98 mm, as shown in Equation 37. The resistance moments are shown in Table 21.

- Mild Steel Bars: l_a = 426.14 mm
- Deformed high-yield Steel Bars: l_a = 402.98 mm

Table 21: Resistance moments as per CP114

Resistance Moments of a Reinforced Concrete Beam with Mild Steel Bars		
M_{r1}	131.1 kNm	$M_{r1} > M_{r2}$
M_{r2}	126.2 kNm	
Resistance Moments of a Reinforced Concrete Beam with High-Yield Steel Bars		
M_{r1}	142.7 kNm	$M_{r1} > M_{r2}$
M_{r2}	126.2 kNm	

For both calculations, the M_{r1} value is larger than that of M_{r2}. Therefore, the maximum design moment must be less than the M_{r2} value. This holds in both cases, as summarised below.

- Mild Steel Bars: $M_{max} = 125.39$ kNm $< M_{r2} = 126.2\ kNm$
- Deformed high-yield Steel Bars: $M_{max} = 125.39$ kNm $< M_{r2} = 126.2\ kNm$

4.5.13. Working Stresses

To determine whether the compressive stress of the concrete σ_c is suitably less than the permissible compressive stress of the concrete due to bending ρ_{ob}, Equation 38 was used. In both cases, the σ_c was significantly less than ρ_{ob}, and therefore the design is sufficient to hold the loads.

Mild Steel design: $\sigma_c = \frac{P_{LF}}{bh} = \frac{125.39kN}{150mm \times 630mm} = 1.33\ MPa$ (Eq. 38)

Deformed high-yield Steel design: $\sigma_c = \frac{P_{LF}}{bh} = \frac{125.39kN}{150mm \times 630mm} = 1.33\ MPa$ (Eq. 38)

Where:
- σ_c is the stress in the concrete.
- P_{LF} is the design force applied to the beam.
- b is the breadth of the beam.
- h is the height of the beam, as per Table 19.

The same concept can be applied to the reinforcement, to ensure that the imposed stresses are less than the permissible tensile stresses ρ_{st} allowed by the code. This is shown in Equation 39. These are both less than the permissible tensile stresses ρ_{st}.

Mild Steel design: $\sigma_s = \frac{P_{LF}}{A_{st}} = \frac{125.39kN}{2198mm^2} = 57.05MPa$ (Equation 39)

Deformed high-yield Steel design: $\sigma_s = \frac{P_{LF}}{A_{st}} = \frac{125.39kN}{1570mm^2} = 79.87MPa$ (Equation 39)

Where:
- σ_s is the stress in the reinforcement.
- P_{LF} is the design force applied to the beam.
- b is the breadth of the beam.
- h is the height of the beam, as per Table 19.

Therefore, the beam did not fail in compression or tension.

4.6. Simply Supported Beam Results: Summary

The results from the simply supported beam designs completed in Sections 4.4 and 4.5 to the Eurocodes and CP114, respectively, are summarised and tabulated in Table 22.

Table 22: Simply supported beam design results summary for the Eurocodes and CP114

Simply Supported Beam Design Results						
Parameter	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild Steel Design**	**CP114: High Yield Steel Design**
Material Parameters						
Design life	years		50			
Exposure class			XC3			
Concrete strength class			C30/37			
Concrete: characteristic cylinder strength	MPa	f_{ck}	30			
Concrete: characteristic cube strength	MPa	$f_{ck,cube}$	37	f_{ck}	30	30
Reinforcement: characteristic strength	MPa	f_{yk}	500	f_y	250	410
Concrete partial factor		γ_C	1.5			
Steel partial factor		γ_S	1.15			
Concrete design strength coefficient		α_{cc}	1			
Concrete design strength	MPa	f_{cd}	20			
Reinforcement design strength	MPa	f_{yd}	434.78			
Concrete density	kN/m³	ρ_C	25	ρ_C	24	24
Permissible Stresses						
Concrete: direct compressive strength	MPa			ρ_{oc}	7.6	7.6
Concrete: Compressive Strength due to Bending	MPa			ρ_{ob}	10	10
Tensile stress in reinforcement Bars (Φ ≤40mm)	MPa			ρ_{st}	140	225.5
Compressive stress in reinforcement bars (Φ ≤40mm)	MPa			ρ_{sc}	125	175
Loads						
Beam self weight	kN	$G_k(beam)$	9	$DL(beam)$	9.36	9.36
Slab self weight	kN	$G_k(slab)$	60	$DL(slab)$	57.60	57.60
Permanent load or dead load	kN	G_k	69	DL	66.96	66.96
Imposed load or live load	kN	$Q_{k,1}$	3	LL	2.70	2.70
Total load	kN	$G_k + Q_k$	72	P	69.66	69.66
Applied factored design load	kN	E_d	97.65	P_{LF}	125.39	125.39
Reaction Forces & Bending Moment						
Reaction force A	kN	$Reac_A$	48.83	$Reac_A$	62.70	62.70
Reaction force B	kN	$Reac_B$	48.83	$Reac_B$	62.70	62.70
Maximum bending moment	kN	M_{max}	97.65	M_{max}	125.39	125.39
Dimensions						
Concrete cover	mm	c_{nom}	35	c	40	40
Breadth	mm	b	150	b	150	150
Length	mm	l	4000	l	4000	4000
Limiting effective depth	mm	d	200	d	200	200
Limiting height	mm	h_{limit}	241	h_{limit}	250	250
Effective depth	mm	$d_{effective}$	379	d_1	580	580
Height	mm	h	420	h	630	630
Calculation Parameters						
Lever arm	mm	z	319	l_a	426.10	402.98
Required reinforcement area	mm²	$A_{s,req}$	704	$A_{st,bal}$	2072	1286
Minimum reinforcement area	mm²	$A_{s,min}$	74	$A_{s,min}$	141.75	141.75
Maximum reinforcement area	mm²	$A_{s,max}$	2274			
Provided reinforcement area	mm²	$A_{s,prov}$	791	$A_{s,prov}$	2198	1570
Provided reinforcement bars			7 x 12mm		7 x 20mm	5 x 20mm
Resistance moment 1	kNm	M_r	109.70	M_{r1}	131.10	142.67
Resistance moment 2	kNm	M_r		M_{r2}	126.15	126.15
Working stress: concrete	MPa			σ_c	1.33	1.33
Working stress: reinforcement	MPa			σ_s	57.05	79.87

4.7. Column Design

A reinforced concrete column, as shown in Figure 17, was designed according to two different structural design codes; the current Eurocodes and the withdrawn Code of Practice CP114. The design considerations and the parameters that were kept constant across both designs are highlighted in *Section 4.8*. *Section 4.9* presents the calculations and results according to the Eurocodes, while *Section 4.10* presents the calculations and results according to CP114. The results of each design were summarised and presented in *Section 4.11*.

4.8. Design Scenario and Constants

The column under consideration is an internal column in a three-story braced structure, as indicated in Figure 17. The factors which were kept constant throughout are described below.

- The column is in a braced structure and has fixed ends.
- It is situated indoors in an office building in Dublin, Ireland.
- The column has a length of 3m.
- The beams spanning in both directions have a breadth of 150mm and a height of 450mm.
- The beam supports regular office loads, its self-weight, and the weight of the 150mm deep slab above it.
- The columns, beams, and slabs are constructed from reinforced concrete.
- The length of the beam on the right-hand side of the column is 6 m, whereas the beam on the left-hand side of the column is 4 m, as shown in Figure 17.

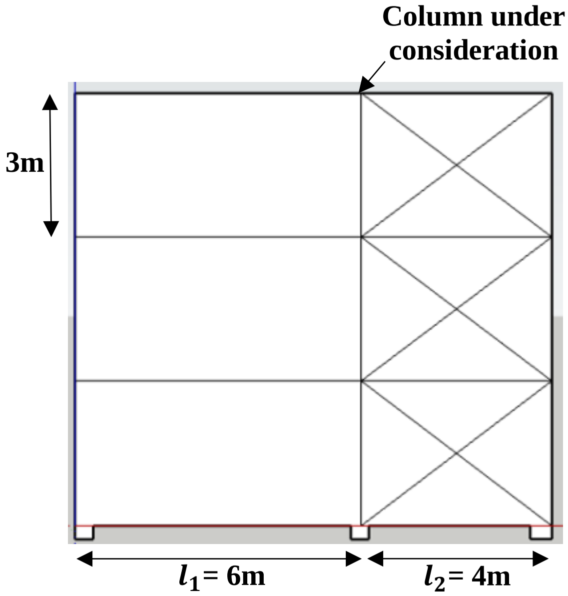

Figure 17: Braced frame and constant parameters (University of Ljubljana, n.d.)

4.9. Eurocodes: Column Design Results

4.9.1. Material Properties

The concrete and steel reinforcement material properties established in the simply supported beam design were also applicable to the following column design. A summary of these values is shown in Table 23.

Table 23: Concrete and reinforcement material properties as per the Eurocodes

Material Property	Symbol	Value	Section Reference
Concrete characteristic cylinder strength	f_{ck}	30 MPa	4.3.1
Concrete characteristic cube strength	$f_{ck,cube}$	37 MPa	4.3.1
Characteristic strength of high-yield steel reinforcement	f_{yk}	500 MPa	4.3.2
Concrete partial factor	γ_C	1.5	4.3.3
Concrete density	ρ_C	25 kN/m³	4.3.4
Concrete design strength	f_{cd}	20 MPa	4.3.3

4.9.2. Loading

4.9.2.1. Permanent Loads

The permanent loads supported by a column are the self-weight of the reinforced concrete beams and slab resting thereon. The self-weight of the beams is determined by Equation 40. The supported slab also imposes a permanent load on the beam, as shown in Equation 41. As shown in Figure 18, the column supports half the span of each beam, plus the slab that rests thereon. The span between frames was specified in *Section 4.8;* the length of the beam on the one side (l_1) is 6 m, and the length of the beam on the other side (l_2) is 4 m.

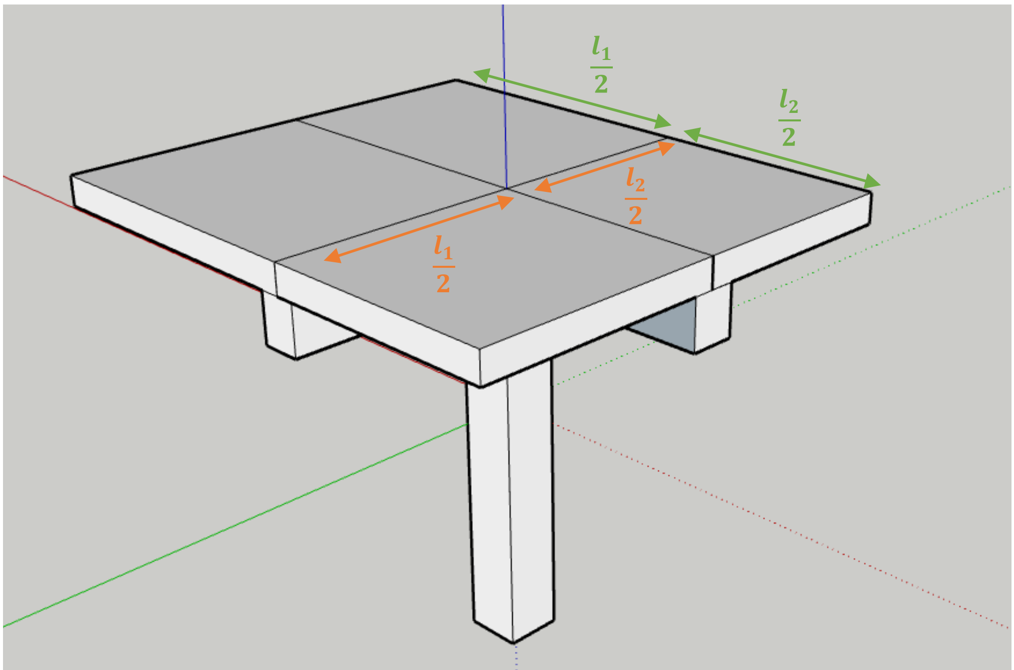

Figure 18: Beam and slab supported by the column

$$G_k(beam) = 2 \times \left(\frac{l_1+l_2}{2}\right) \times h \times \rho_c = 2 \times 5 \times 0.45 \times 25 = 112.5\ kN/m \quad \text{(Equation 40)}$$

$$G_k(slab) = \left(\frac{l_1+l_2}{2}\right) \times d_{slab} \times \rho_c = 5 \times 0.15 \times 25 = 18.75\ kN/m \quad \text{(Equation 41)}$$

Where:

- $G_k(beam)$ is the self-weight of the beam, calculated as a force per metre length of the beams in the frame.
- $G_k(slab)$ is the self-weight of the slab, calculated as a force per metre length of the beams in the frame.
- l_1 is the longer beam span on the one side of the column, which is a controlled variable from *Section 4.8*.
- l_2 is the shorter beam span on the other side of the column, which is a controlled variable from *Section 4.8*.
- h is the beam height, which is a controlled variable from *Section 4.8*.
- b is the beam breadth, which is a controlled variable from *Section 4.8*.
- ρ_c is the concrete density, as determined in Table 23.
- l_1 is the length of the beam on the left-hand side of the column, which is a controlled variable from *Section 4.8*.
- l_2 is the length of the beam on the right-hand side of the column, which is a controlled variable from *Section 4.8*.
- d_{slab} is the depth of the supported slab, which is a controlled variable from *Section 4.8*.

Therefore, the sum of the self-weights of the beams and the supported slab gives a permanent load G_k of 131.25 kN/m.

4.9.2.2. Imposed Loads

The column under consideration is in an office building. Category B: Office Areas was chosen as the usage category for this design scenario. The imposed loads for office areas are summarised in Table 13. The average of the minimum and maximum values was multiplied by the span of the supported slab to determine the load acting on the column. Therefore, $Q_{k,1}$ is taken as 12.5 kN/m. Additionally, all finishes have been ignored as part of this research.

4.9.3. Design Loads

The maximum design load using the partial factors (γ) from the most unfavourable scenario of the ULS is shown in Equation 42. E_d represents the design load at each floor per metre length of the beam.

$$E_d = 1.35G_{k,j} + 1.5Q_{k,1} = 195.94\ kN/m \quad \text{(Equation 42)}$$

Where:

- E_d is the design load at each floor of the building per metre length of the beam.
- $G_{k,j}$ is the permanent load calculated in *Section 4.9.2*.
- $Q_{k,1}$ is the temporary load from *Section 4.9.2*.

To determine the axial load acting on each column in the structure of Figure 17, and not the distributed load, acting per metre length of the beam, E_d was multiplied by half the length of each of the beams which it supports in the frame, as shown in Equations 81 to 83 below.

$N_{Ed,3} = E_d \times \left(\frac{l_1+l_2}{2}\right) = 979.69\ kN$ (Equation 81)

$N_{Ed,2} = E_d \times \left(\frac{l_1+l_2}{2}\right) + N_{Ed,3} = 1959.38\ kN$ (Equation 82)

$N_{Ed,1} = E_d \times \left(\frac{l_1+l_2}{2}\right) + N_{Ed,2} = 2939.06\ kN$ (Equation 83)

Where:
- $N_{Ed,x}$ is the axial load at the specified column.
- E_d is the design load at each floor of the building per metre length of the beam.
- l_1, l_2 are the lengths of the beams on either side of the column.

4.9.4. Short versus Slender Columns

The slenderness ratio λ and the slenderness limit λ_{lim} were determined using the steps in *Section 3.9.4.*

Several iterations of the column design were used to establish the required cross-sectional dimensions of the column. All the iterations are not described here; however, the optimal required dimensions are shown in Figure 19 and were determined based on iterative calculations using the equations from *Sections 3.9.4 to 3.9.6*.

After several column design iterations, the optimal column cross-section dimensions were determined to be 400 mm by 425 mm. This was determined by iteratively changing the dimensions of the column to establish at which dimensions it would fail. At 400mm by 425mm, the column could still withstand the load and complied with all the checks required for column design, such as:

- The slenderness ratio remains less than the slenderness limit, to ensure that the column is short.
- The required reinforcement area required to withstand the load applied to the column was determined using the column reinforcement design charts, as per Section 4.9.5.

Figure 19 shows the dimensions as well as the axis of the cross-section. The slenderness check was only considered for the smallest dimension as it was assumed that if the column was to buckle, it would buckle about the X-X axis because it has a higher slenderness ratio about this axis.

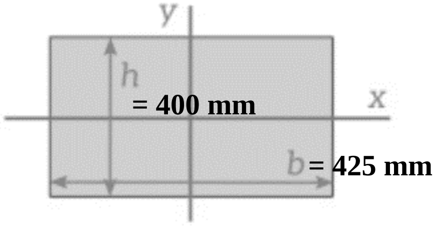

Figure 19: Column cross-section with dimensions

4.9.4.1. Area Moment of Inertia

The area moment of inertia for the column cross-section about the x-axis was calculated using Equation 43. The area moment of inertia of the beams in the structure was also calculated, as per Equation 43.

$$I_{col} = \frac{b_{col}h_{col}{}^{3}}{12} = \frac{425\times400^{3}}{12} = 2\,266\,666\,667\ mm^{4} \qquad \text{(Equation 43)}$$

$$I_{beam} = \frac{b_{beam}h_{beam}{}^{3}}{12} = \frac{450\times150^{3}}{12} = 126\,562\,500\ mm^{4} \qquad \text{(Equation 43)}$$

Where:

- I_{col} is the area moment of inertia of the column about the X-X axis.
- I_{beam} is the area moment of inertia of the beam as it bends under a load.
- b_{col} is the breadth of the column as shown in Figure 19.
- h_{col} is the height of the column as shown in Figure 19**.**
- b_{beam} is the breadth of the beam, which is a controlled variable from *Section 4.8*.
- h_{beam} is the height of the beam, which is a controlled variable from *Section 4.8*.

4.9.4.2. Stiffness

The relative stiffness of the column ends was calculated using Equation 44. The columns and the beams are made of the same material, therefore the elastic modulus (E) is cancelled out in the fraction. k_1, k_2

$$k_1 = k_2 = \frac{k_{col}}{\sum k_{beam}} = \frac{\left({}^{EI}/_{L}\right)_{col}}{2\sum\left({}^{EI}/_{L}\right)_{beam}} = 7.462 \qquad \text{(Equation 44)}$$

Where:

- k_1, k_2 are the relative stiffnesses at the top and bottom ends of the column.
- E is the elastic modulus of the element, which is cancelled out in Equation 44 as the beams are made of the same material.
- I_{col} is the area moment of inertia of the column, as calculated in *Section 4.9.4.1*.
- I_{beam} is the area moment of inertia of the beams, as calculated in *Section 4.9.4.1*.
- L_{col} is the length of the column, which is a controlled variable from *Section 4.8*.
- $\mathrm{L_{beam}}$ is the span between frames, which is a controlled variable from *Section 4.8*.

4.9.4.3. Radius of Gyration

Equation 46 was used to determine the radius of gyration of the column.

$$i = \sqrt{\frac{I}{A}} = \sqrt{\frac{I_{col}}{bh}} = 115.47mm \qquad \text{(Equation 46)}$$

Where:

- i is the radius of gyration of the columns.
- I_{col} is the area moment of inertia around the X-axis, as per *Section 4.9.4.1*.
- b is the breadth of the column as shown in Figure 19.
- h is the height of the column as shown in Figure 19.

4.9.4.4. Effective Column Length

The effective length of the column, which is situated in a regular braced frame, was determined using Equation 47.

$$l_0 = 0.5l\sqrt{\left(1+\frac{k_1}{0.45+k_1}\right)\left(1+\frac{k_2}{0.45+k_2}\right)} = 2.92\ m \qquad \text{(Equation 47)}$$

Where:
- l_0 is the effective length of each of the columns.
- l is the length of the column, which is a controlled variable from *Section 4.8*.
- k_1, k_2 are the relative stiffnesses at ends 1 and 2 respectively, as per *Section 4.9.4.2*.

4.9.4.5. Slenderness Ratio

The slenderness ratio for each of the three columns would be the same value, as they have the same effective lengths. The slenderness ratio was determined using Equation 48.

$$\lambda = \frac{l_0}{i} = 25.24 \qquad \text{(Equation 48)}$$

Where:
- λ is the slenderness ratio of the column.
- l_0 is the effective length of the column, as determined in *Section 4.9.4.4*.
- i is the radius of gyration of the column, as determined in *Section 4.9.4.3*.

4.9.4.6. Slenderness Limit

The simplified Equation 55 from *Section 3.6.4* was used in this design scenario due to the unknown values for creep and the mechanical reinforcement ratio.

$$\lambda_{lim} = \frac{26.2}{\sqrt{\frac{N_{Ed}}{A_c f_{cd}}}} = 28.18 \qquad \text{(Equation 55)}$$

Where:
- $\lambda_{lim}$is the slenderness limit for the column under consideration.
- N_{Ed} is the maximum axial load, which was determined in *Section 4.9.3*.
- A_c is the area of concrete in the cross-section of the column, as shown in Figure 19.
- f_{cd} is the design strength of the concrete, as shown in Table 23.

From the slenderness ratio and the slenderness limits, it is evident that the column is short, as shown in Equation 84.

$$\lambda = 25.24 < \lambda_{lim} = 28.18 \therefore column\ is\ short \qquad \text{(Equation 84)}$$

4.9.5. Column Reinforcement

The cross-sectional area of the reinforcement required in the columns can be estimated from the rectangular column charts which have been created based on EN1992. An example of such a chart is shown in Figure 20. These charts may only be used for reinforcement placed symmetrically in a column cross-section and concentrated in the corners. These charts may also only be used if the characteristic concrete strength (f_{ck}) is less than 50MPa, and the characteristic strength of steel reinforcement (f_{yk}) is less than 500MPa.

The rectangular column chart was chosen based on the $^{d_2}/_h$ ratio shown in Figure 20. d_2 was determined based on the concrete cover requirements from EN1992, which are discussed in

Section 3.3.11 and was taken as 43mm as shown in Equation 85. The ${d_2}/{h}$ ratio is shown in Equation 86 and was determined to be 0.11. Therefore, the 0.1 rectangular column chart was used, as shown in Figure 20.

$$d_2 = c_{min} + \Delta c_{dev} + \frac{\Phi_{rebar}}{2} = 25 + 10 + \frac{20}{2} = 45\ mm \qquad \text{(Equation 85)}$$

$$\frac{d_2}{h} = \frac{45}{400} = 0.11 \approx 0.1 \qquad \text{(Equation 86)}$$

Where:
- d_2 is the depth to the centroid of the bars from the surface of the concrete.
- c_{min} is the minimum cover to the reinforcement as determined from *Section 3.3.11*.
- Δc_{dev} is the allowance for deviation and is recommended as 10mm, as per *EN1992*.
- Φ_{rebar} is the diameter of the reinforcement, which is assumed to be 20mm for this column design.
- h is the height of the member perpendicular to the axis under consideration.

The applied axial load (N_{Ed}) and the applied bending moment (M_{Ed}) are required to determine the cross-sectional area of reinforcement (A_s), as shown in Figure 20. N_{Ed} was determined in *Section 4.9.3*. However, the fixed end moments (FEM) for each column were calculated using Equations 87 and 88.

$$FEM_{beam\ 1} = \frac{wl^2}{12} = 587.8kNm \qquad \text{(Equation 87)}$$

$$FEM_{beam\ 2} = \frac{wl^2}{12} = 261.3kNm \qquad \text{(Equation 88)}$$

Where:
- $FEM_{beam\ x}$ is the bending moment of each beam respectively.
- w is the design load E_d, as determined in *Section 4.9.3*.
- l is the length of beam 1 or beam 2, respectively, which are controlled variables from *Section 4.8*.

The input values for the X and Y axes of the chart in Figure 20 are tabulated in Table 24. These values are based on Equations 56 to 62. The quantity of required reinforcement ($A_{s,req}$) as determined from Figure 20 is also tabulated in Table 24. The required reinforcement for the middle floor column is annotated on Figure 20.

$$k_{col} = \left({I}/{L}\right)_{col} = 0.75 \times 10^{-3} \qquad \text{(Equation 56)}$$

$$k_{beam,1} = \left({I}/{L}\right)_{beam,1} = 0.09 \times 10^{-3} \qquad \text{(Equation 57)}$$

$$k_{beam,2} = \left({I}/{L}\right)_{beam,2} = 0.14 \times 10^{-3} \qquad \text{(Equation 58)}$$

$$\sum k = k_{col,u} + k_{col,l} + k_{beam,1} + k_{beam,2} \qquad \text{(Equation 59)}$$

$$\Delta k = \frac{k_{col}}{\sum k} \qquad \text{(Equation 60)}$$

$$\Delta M = |FEM_{beam1} - FEM_{beam2}| \quad \text{(Equation 61)}$$

$$M_{Ed} = \Delta k \times \Delta M \quad \text{(Equation 62)}$$

Where:

- k_{col} is the stiffness of the column in question.
- $k_{beam,1}$, $k_{beam,2}$ is the stiffness of beams 1 and 2, respectively, where beams 1 and 2 are on either side of the column in the frame.
- I is the area moment of inertia, as per *Section 4.9.4.1*.
- L is the length of beam 1 or 2, respectively.
- $k_{col,u}$, $k_{col,l}$ is the stiffness of the upper column and lower column respectively. For the topmost floor, $k_{col,u}$ was taken as 0 as there was no column above it.
- Δk is the effective stiffness.
- ΔM is the difference between the fixed end moments of the beams, as per Equations 87 and 88.
- M_{Ed} is the bending moment acting on the column under consideration.

Table 24: Column reinforcement design chart inputs and results

Fl.	Stiffness Σk	Effective Stiffness Δk	Moment Difference ΔM (kNm)	Bending Moment M_{Ed} (kNm)	Axial Load N_{Ed} (kN)	X-Axis $\frac{M_{Ed}}{bh^2 f_{ck}}$	Y-Axis $\frac{N_{Ed}}{bhf_{ck}}$	$\frac{A_s f_{yk}}{bhf_{ck}}$	$A_{s,req}$ mm²
3rd	0.993	0.761	326.6	248.51	980	0.122	0.19	**0.18**	1836
2nd	1.748	0.432	326.6	141.12	1960	0.069	0.38	**0.05**	510
1st	1.748	0.432	326.6	141.12	2940	0.069	0.58	**0.22**	2244

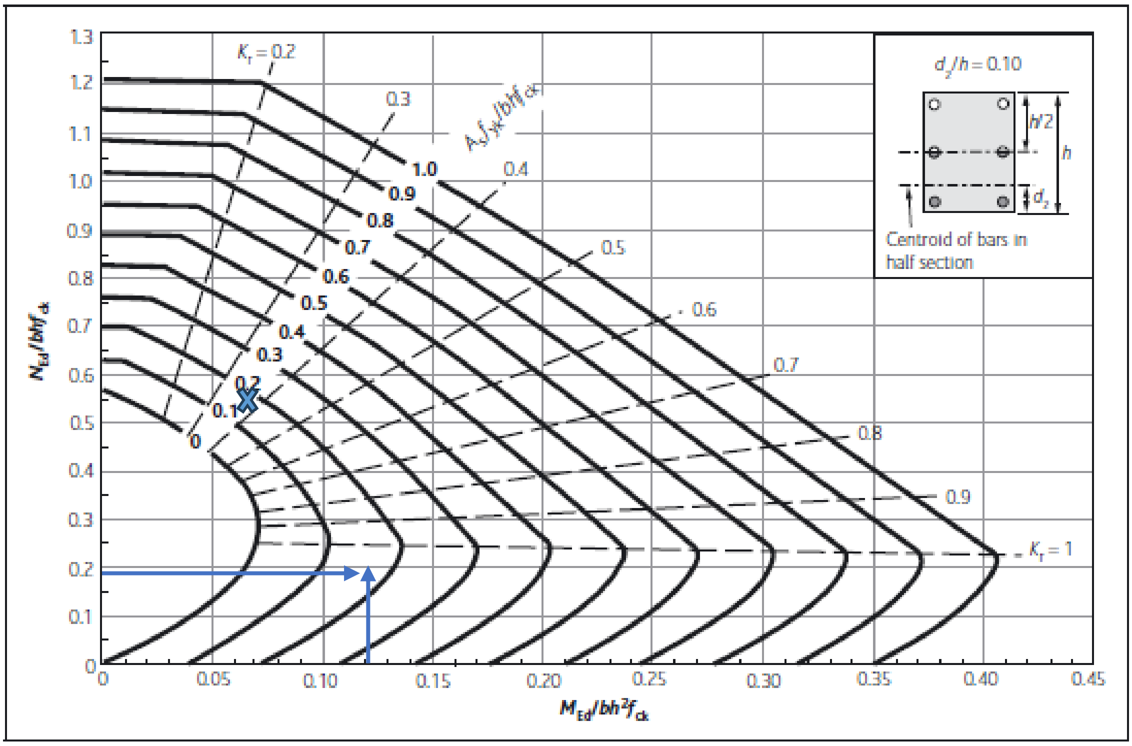

Figure 20: Column design chart for rectangular columns (Goodchild, 2009)

4.9.6. Minimum and Maximum Reinforcement

The minimum and maximum quantities of reinforcement should be checked against the required reinforcement ($A_{s,req}$) from *Section 4.9.5*. The provided reinforcement area should be larger than the value calculated from Equation 63. The provided reinforcement should also be less than 4% of the column's cross-sectional area. The required, minimum, maximum, and provided reinforcement areas are shown in Table 25.

$$A_{s,min,1} = \frac{0.1N_{Ed}}{0.87f_{yk}} \; or \; A_{s,min,2} = 0.002A_c, whichever\ is\ greater \qquad \text{(Equation 63)}$$

Where:
- $A_{s,min,x}$ is the minimum cross-sectional area of reinforcement.
- N_{Ed} is the axial load at the specified column, as per *Section 4.9.3*.
- f_{yk} is the characteristic strength of high-yield steel reinforcement, as per Table 23.
- A_c is the cross-sectional area of the reinforced concrete column, as per Figure 19.

Table 25: The minimum, maximum, required, and provided cross-sectional reinforcement areas for the column

Floor	Minimum Reinforcement $A_{s,min,1}$	Minimum Reinforcement $A_{s,min,2}$	Maximum Reinforcement $A_{s,max}$	Required Reinforcement $A_{s,req}$	Provided Reinforcement $A_{s,prov}$
3rd	225 mm²	340 mm²	6800 mm²	1836 mm²	1884 mm²
2nd	450 mm²	340 mm²	6800 mm²	510 mm²	628 mm²
1st	676 mm²	340 mm²	6800 mm²	2244 mm²	2512 mm²

4.10. Code of Practice CP114: Column Design Results

4.10.1. Material Properties

The concrete and steel reinforcement material properties established in the simply supported beam design were also applicable to the following column design. A summary of these values is shown in Table 26, and a reference to the corresponding sections is required for further information. The permissible stresses in the concrete and the reinforcement are also presented in Table 26, with the relevant section reference.

Table 26: Concrete and steel reinforcement material properties as per CP114

Material Property	Symbol	Value	Section Reference
Concrete cube compressive strength	f_{ck}	30 MPa	4.4.1
Mild reinforcement yield strength	f_y	250 MPa	4.4.2
Hot-rolled deformed reinforcement yield strength	f_y	410 MPa	4.4.2
Concrete density	ρ_C	24 kN/m³	4.4.3
Permissible Stresses in the Concrete	**Symbol**	**Value**	**Section Reference**
Direct compressive strength	ρ_{oc}	7.6 MPa	4.4.9
Compressive strength due to bending	ρ_{ob}	10 MPa	4.4.9
Permissible Stresses in the Reinforcement	**Symbol**	**Value**	**Section Reference**
Tensile stress in mild steel bars ($\Phi \leq$40mm)	ρ_{st}	140 MPa	4.4.9
Compressive stress in mild steel bars ($\Phi \leq$40mm)	ρ_{sc}	125 MPa	4.4.9
Tensile stress in deformed, high-yield bars ($\Phi \leq$20mm)	ρ_{st}	225.5 MPa	4.4.9
Compressive stress in deformed, high-yield bars ($\Phi \leq$20mm)	ρ_{sc}	175 MPa	4.4.9

4.10.2. Loading

4.10.2.1. Dead Loads

The self-weights of the beams and slab supported by the column are determined by Equations 64 and 65, respectively. The beam and slab supported by the column are shown in Figure 18. Therefore, the dead load is the sum of the self-weights of the beam and the supported slab. Therefore, the sum of the self-weights of the beam and the supported slab gives a permanent load DL of 126 kN/m.

$$DL(beam) = 2 \times \left(\frac{l_1+l_2}{2}\right) \times h \times \rho_c = 2 \times 5 \times 0.45 \times 24 = 108\ kN/m \qquad \text{(Equation 64)}$$

$$DL(slab) = \left(\frac{l_1+l_2}{2}\right) \times d_{slab} \times \rho_c = 5 \times 0.15 \times 24 = 18\ kN/m \qquad \text{(Equation 65)}$$

Where:
- $DL(beam)$ is the self-weight of the beam, calculated as a force per metre length of the beams in the frame.
- $DL(slab)$ is the self-weight of the slab, calculated as a force per metre length of the beams in the frame.
- $span$ is the span between the frames of the building.
- h is the beam height.
- ρ_c is the concrete density, as per Table 23.

4.10.2.2. Live Loads

A summary of the live loads for "Category B: offices for general use" is in Table 17. The distributed live load was multiplied by the span between frames, a constant given in *Section 4.8*, to determine the live load per metre length of the beam as 12.5 kN/m.

4.10.2.3. Loading Summary

The overall loading per CP114 is the sum of the dead loads and live loads, as shown in Equation 29, which is then multiplied by the load factor, as shown in Equation 30.

$$P = DL + LL = 138.5\ kN/m \qquad \text{(Equation 29)}$$

$$P_{LF} = 1.8 \times P = 249.3\ kN/m \qquad \text{(Equation 30)}$$

To determine the axial load acting on each column in the structure of Figure 17, and not the load acting per metre length of the beam, P_{LF} was multiplied by half the length of each of the beams which it supports in the frame, as shown in Equations 89 to 91 below.

$$N_{PLF,3} = P_{LF} \times \left(\frac{l_1+l_2}{2}\right) = 1246.5\ kN \qquad \text{(Equation 89)}$$

$$N_{PLF,2} = P_{LF} \times \left(\frac{l_1+l_2}{2}\right) + N_{PLF,3} = 2493\ kN \qquad \text{(Equation 90)}$$

$$N_{PLF,1} = P_{LF} \times \left(\frac{l_1+l_2}{2}\right) + N_{PLF,2} = 3739.5\ kN \qquad \text{(Equation 91)}$$

Where:
- $N_{PLF,x}$ is the axial load at the specified column.
- P_{LF} is the design load at each floor of the building per metre length of the beam, as per Equation 30.
- l_1, l_2 are the lengths of the beams on either side of the column.

4.10.3. Short versus Long Columns

Several iterations of the column design were used to establish the required cross-sectional dimensions of the column. All the iterations are not described here; however, the optimal required dimensions are shown in Figures 21 and 22, depending on the type of reinforcing steel used. The dimensions were determined based on iterative calculations using the equations from *Sections 3.10.3 to 3.10.5.*

The optimal column cross-section dimensions were determined to be 675 mm by 700 mm if mild steel reinforcement was used, and 650 mm by 700mm if high-yield reinforcement was used. This was determined by iteratively changing the dimensions of the column to establish at which dimensions it would fail. At these dimensions, the column could still withstand the load and complied with all the checks required for column design, such as:

- The slenderness ratio remains less than the slenderness limit of 15, to ensure that the column is short, as per Equation 66 below.
- The column is controlled by compression, as per Section 4.10.5.
- The limiting load of the beam is more than the maximum load applied to the beam, therefore it will not fail in compression, as per Section 4.10.5.

Figures 21 and 22 show the dimensions as well as the axis of the cross-section for the mild steel design versus the high-yield steel design. The slenderness check was only checked for the smallest dimension as it was assumed that if the column was to buckle, it would buckle about the X-X axis because it has a higher slenderness ratio about this axis.

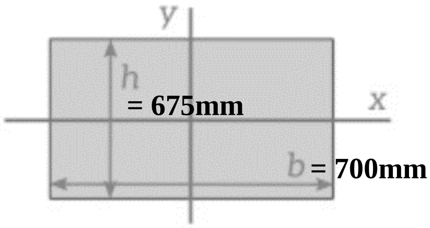

Figure 21: Column cross-section for a design with mild steel reinforcement

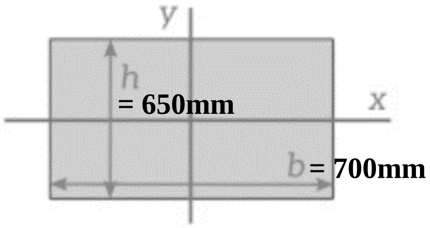

Figure 22: Column cross-section for a design with deformed high-yield steel reinforcement

The slenderness ratios of the columns in Equation 66 for columns using mild steel or high-yield steel respectively. The effective column length is determined from *Table 19 in CP114*. It was determined that both columns are short.

$$\lambda_{mild} = \frac{l_0}{b} = \frac{0.75l}{b} = 3.33 < 15 \;\therefore\; column\ is\ short \qquad \text{(Equation 66)}$$

$$\lambda_{high\ yield} = \frac{l_0}{b} = \frac{0.75l}{b} = 3.46 < 15 \;\therefore\; column\ is\ short \qquad \text{(Equation 66)}$$

Where:
- λ is the slenderness ratio.
- l_0 is the effective length of the column.
- l is the length of the column, given as a controlled variable in *Section 4.8*.
- b is the smallest lateral dimension of the cross-section of the column, as per Figures 21 and 22.

4.10.4. Required Reinforcement

The area of the reinforcement required in the element's cross-section per Equation 69 is shown below for columns using mild steel or high-yield steel.

$0.8\%A_c = 3780\ mm^2 > A_{s,req} < 8\%A_c = 37800\ mm^2$ (Equation 69)

$0.8\%A_c = 3640\ mm^2 > A_{s,req} < 8\%A_c = 36400\ mm^2$ (Equation 69)

Where:
- $A_{s,req}$ is the area of cross-sectional reinforcement required in the column.
- A_c is the cross-sectional area of the concrete column.

The required reinforcement area if mild steel is used is between 3780 mm² and 37800 mm², whereas if high-yield steel is used, the required reinforcement area is between 3640 mm² and 36400 mm². The provided area of reinforcement for each case was determined after several iterations using Equations 70 to 74. The provided reinforcement should ensure that the design load P is larger than the limiting load P_b and that the design load P is less than the permissible load P_o. These are explained in *Section 4.10.5.* and the optimal provided reinforcement is summarised below.

- Provided mild steel reinforcement area = 4068 mm² (9 x 24mm diameter bars)
- Provided deformed high-yield reinforcement area = 4068 mm² (9 x 24mm diameter bars)

4.10.5. Design for Short Columns Subject to Direct Load and Bending

The column cross-section was determined to be controlled by compression because the load $N_{PLF,1}$, as determined in *Section 4.10.2*, exceeded the limiting load P_b, as per Equation 70 for columns using mild steel or high-yield steel respectively.

$P_{b,mild} = \rho_{oc} b d_1 X - A_{sc}(\rho_{st} - \rho_{sc}) = 167.7\ kN$ (Equation 70)

$P_{b,high\ yield} = \rho_{oc} b d_1 X - A_{sc}(\rho_{st} - \rho_{sc}) = 167.7\ kN$ (Equation 70)

Where:
- P_b is the limiting load.
- ρ_{oc} is the permissible stress of the concrete in direct compression, as per Table 23.
- d_1 is the effective depth to the tensile reinforcement, taken as twice the reinforcement diameter plus half of the reinforcement diameter, to reach the effective depth.
- A_{sc} is the area of the compressive reinforcement, which for the conditions of bending is equal to half of the total area of reinforcement in the column.
- ρ_{st} is the permissible tensile stress in the reinforcement, as per *Section 3.7.1*.
- ρ_{sc} is the permissible compressive stress in the reinforcement, as per *Section 3.7.1*
- X is a factor used in the determination of the limiting load, and is shown in Equations 72 and 73, respectively for mild steel and high-yield steel, where E_s is the secant modulus of elasticity of steel at a stress of $1.8\rho_{st}$.

$X = \frac{585}{690+1.8\rho_{st}} = 0.621$ (Equation 72)

$X = \frac{585}{690+\frac{375\rho_{st}\times10^3}{E_s}} = 0.847$ (Equation 73)

Therefore, the load $N_{PLF,1}$, as determined in *Section 4.10.2*, was related to the permissible load P_o for an axially loaded column and did not exceed the value given in Equation 74 for columns using mild steel or high-yield steel respectively.

$$P_{o,mild} = \rho_{oc}A_c + \rho_{sc}A_{sc} = 3814.3\ kN \qquad \text{(Equation 74)}$$

$$P_{o,high\ yield} = \rho_{oc}A_c + \rho_{sc}A_{sc} = 3783.0\ kN \qquad \text{(Equation 74)}$$

Where:

- P_o is the permissible axial load.
- ρ_{oc} is the permissible compressive stress in the reinforcement, as per *Section 3.7.1*
- A_c is the cross-sectional area of concrete, minus the reinforcement area.
- ρ_{sc} is the permissible compressive stress in the reinforcement, as per *Section 3.7.1*
- A_{sc} is the cross-sectional area of the longitudinal steel.

4.11. Column Results Summary

The results from the column designs in Sections 4.9 and 4.10 are tabulated in Table 27.

Table 27: Column design results

Column Design Results						
Material Properties	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Concrete: characteristic cylinder strength	MPa	f_{ck}	30			
Concrete: characteristic cube strength	MPa	$f_{ck,cube}$	37	f_{ck}	30	30
Reinforcement: characteristic strength	MPa	f_{yk}	500	f_y	250	410
Concrete density	kN/m³	ρ_C	25	ρ_C	24	24
Concrete partial factor		γ_C	1.5			
Concrete design strength	MPa	f_{cd}	20			
Concrete: permissible direct compressive stress	MPa			ρ_{oc}	7.6	7.6
Concrete: compressive stress due to bending	MPa			ρ_{ob}	10	10
Reinforcement: permissible compressive stress	MPa			ρ_{sc}	125	175
Reinforcement: permissible tensile stress	MPa			ρ_{st}	140	225.5
Dimensions	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Column length	m	l	3	l	3	3
Column cross-section breadth	mm	b	400	b	675	650
Column cross-section height	mm	h	425	h	700	700
Loading	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Permanent load from supported beams	kN/m	$G_k(beam)$	112.5	$DL(beam)$	108	108
Permanent load from supported slab	kN/m	$G_k(slab)$	18.75	$DL(slab)$	18	18
Total permanent Load	kN/m	G_k	131.25	DL	126	126
Variable load	kN/m	Q_k	12.5	LL	12.5	12.5
Total design load		$G_k + Q_k$	143.75	P	138.5	138.5
Factored design load	kN/m	E_d	195.94	P_{LF}	249.3	249.3
Load acting on 3rd floor column	kN	$N_{Ed,3}$	979.69	$N_{PLF,3}$	1246.5	1246.5
Load acting on 2nd floor column	kN	$N_{Ed,2}$	1959.38	$N_{PLF,2}$	2493	2493
Load acting on 1st floor column	kN	$N_{Ed,1}$	2939.06	$N_{PLF,1}$	3739.5	3739.5
Effective Length	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Column area moment of inertia	mm⁴	I_{col}	2266666667			
Beam area moment of inertia	mm⁴	I_{beam}	126562500			
Relative stiffness		k_1, k_2	7.462			
Effective height	m	l_0	2.920	l_0	2.25	2.25
Radius of gyration	mm	i	115.47			
Slenderness ratio		λ	25.24	λ	3.33	3.46
Slenderness limit		λ_{lim}	28.18	λ_{lim}	15	15
Slender or short column			**SHORT**		**SHORT**	**SHORT**
Fixed End Moments (FEM)	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
FEM in beam 1	kNm	$FEM_{beam\ 1}$	587.80			
FEM in beam 2	kNm	$FEM_{beam\ 2}$	261.30			

Column Design Results - continued						
Reinforcement	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Cover to reinforcement	mm	c_{min}	25			
Additional cover (deviation allowance)	mm	Δc_{dev}	10			
Reinforcement diameter	mm	Φ_{rebar}	20	Φ_{rebar}	24	24
Depth to centroid of reinforcement	mm	d_2	45			
CP114: Permissible Loads	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Factor X				X	0.621	0.847
Effective depth to tensile reinforcement	mm			d_1	60	60
Limiting load	kN			P_b	167.70	167.65
Permissible axial load	kN			P_o	3814.3	3783.03
Eurocodes Reinforcement Chart	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Choose reinforcement chart	**USE CHART 0.1**		**0.11**			
Reinforcement chart (Asfyk)/(bhfck): 3rd floor			0.18			
Reinforcement chart (Asfyk)/(bhfck): 2nd floor			0.05			
Reinforcement chart (Asfyk)/(bhfck): 1st floor			0.22			
Required reinforcement area: 3rd floor	mm²	$A_{s,req}$	1836			
Required reinforcement area: 2nd floor	mm²	$A_{s,req}$	510			
Required reinforcement area: 1st floor	mm²	$A_{s,req}$	2244			
Minimum Reinforcement	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Minimum reinforcement: 3rd floor	mm²	$A_{s,min}$	225.00	$A_{s,min}$	3780	3640
Minimum reinforcement: 2nd floor	mm²	$A_{s,min}$	340.00	$A_{s,min}$	3780	3640
Minimum reinforcement: 1st floor	mm²	$A_{s,min}$	340.00	$A_{s,min}$	3780	3640
Maximum Reinforcement	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Maximum reinforcement: 3rd floor	mm²	$A_{s,max}$	6800	$A_{s,max}$	37800	36400
Maximum reinforcement: 2nd floor	mm²	$A_{s,max}$	6800	$A_{s,max}$	37800	36400
Maximum reinforcement: 1st floor	mm²	$A_{s,max}$	6800	$A_{s,max}$	37800	36400
Provided Reinforcement	**Units**	**Symbol**	**Eurocode Design**	**Symbol**	**CP114: Mild steel design**	**CP114: High yield steel design**
Provided reinforcement: 3rd floor	mm²	$A_{s,prov}$	1884.00	$A_{s,prov}$	4068	4068
Provided reinforcement: 2nd floor	mm²	$A_{s,prov}$	628.00	$A_{s,prov}$	4068	4068
Provided reinforcement: 1st floor	mm²	$A_{s,prov}$	2512.00	$A_{s,prov}$	4068	4068

4.12. Embodied Carbon Results

4.12.1. Dimensions and Material Quantities

To determine the embodied carbon from each of the column and beam designs in this research project, the volume of concrete and steel reinforcement required by each design was determined. These quantities are required as inputs for embodied carbon calculator. The dimensions of the beam and column designs have been tabulated in Tables 28 and 29, for the simply supported beams and columns respectively.

Table 28: Simply supported beam dimensions for each design case

Dimension	Units	Eurocodes	CP114 – Mild Steel	CP114 – High Yield Steel
Length l	mm	4000	4000	4000
Breadth b	mm	150	150	150
Height h	mm	420	630	630
Reinforcement Area $A_{s,prov}$	mm²	791	2198	1570

Table 29: Column dimensions for each design case

Dimension	Units	Eurocodes	CP114 – Mild Steel	CP114 – High Yield Steel
Length l	mm	3000	3000	3000
Breadth b	mm	400	675	650
Height h	mm	425	700	700
Reinforcement Area: 3rd floor $A_{s,prov}$	mm²	1884	4068	4068
Reinforcement Area: 2nd floor $A_{s,prov}$	mm²	628	4068	4068
Reinforcement Area: 1st floor $A_{s,prov}$	mm²	2512	4068	4068

4.12.1.1. Concrete Volumes

The following concrete volumes apply to each structural element design based on the dimensions in Tables 28 and 29.

Beams:

- Eurocodes: 0.252 m³ of concrete.
- CP114 – Mild Steel Design: 0.378 m³ of concrete.
- CP114 – High Yield Steel Design: 0.378 m³ of concrete

Columns:

- Eurocodes: 1.53 m³ of concrete over three floors.
- CP114 – Mild Steel Design: 4.253 m³ of concrete over three floors.
- CP114 – High Yield Steel Design: 4.095 m³ of concrete over three floors.

4.12.1.2. Reinforcement Volumes

The volumes of reinforcement in the columns and beams were calculated by multiplying the reinforcement cross-sectional area by the length of the column or beam. Using the values in Tables 28 and 29, the reinforcement volumes are shown below.

Beams:

- Eurocodes: 0.00316 m^3 of steel.
- CP114 – Mild Steel Design: 0.00879 m^3 of steel.
- CP114 – High Yield Steel Design: 0.00628 m^3 of steel.

Columns:

- Eurocodes: 0.0151 m^3 of steel over three floors.
- CP114 – Mild Steel Design: 0.0366 m^3 of steel over three floors.
- CP114 – High Yield Steel Design: 0.0366 m^3 of steel over three floors.

4.12.1.3. Concrete Volume less the Reinforcement

When accounting for the steel within the concrete, the overall volumes of concrete and steel that were provided for each element are shown in Tables 30 and 31 for the simply supported beams and columns, respectively.

Table 30: Material quantities required in each simply supported beam design case

Material Quantities	Units	Eurocodes	CP114 – Mild Steel	CP114 – High Yield Steel
Concrete	m^3	0.249	0.369	0.372
Reinforcement	m^3	0.00316	0.00879	0.00628

Table 31: Material quantities required in each column design case

Material Quantities	Units	Eurocodes	CP114 – Mild Steel	CP114 – High Yield Steel
Concrete	m^3	1.515	4.216	4.059
Reinforcement	m^3	0.0151	0.0366	0.0366

4.12.2. Embodied Carbon Calculations

The embodied carbon of the beams and columns designed as part of this research project was calculated using the Embodied Carbon Calculator for Concrete (Circular Ecology, 2019). The embodied carbon for one cubic metre of each mix was calculated. This was then proportioned to identify the exact embodied carbon of the beam or column. The procedure to determine inputs to the calculator as well as all results are discussed below.

4.12.2.1. Concrete Mixture Inputs

The inputs are summarised in Table 32, and each variable is described below. In the software, the option for the concrete to be placed in situ, rather than precast, was chosen, as this is the most common method of manufacturing concrete in industry.

Cement type:
The cement type was kept constant across all mixtures to ensure that an accurate comparison could be made. It was also decided that no SCMs would be included in the mixtures, to ensure that purely the unintentional effects of the codes on the quantities of required materials could be compared. Therefore, CEM I Portland cement was used.

Cement quantity:
The Eurocodes refer to the European Standard *EN206-1 Concrete – Part 1: Specification, Performance, Production and Conformity*. The minimum quantity of cement is prescribed by this code depending on the concrete strength class, design life, and exposure class of the intended element. Roadstone, a concrete supplier in Ireland, published a guide intended to aid designers in efficiently designing a concrete mix to *EN206-1*. From this guide, a minimum cement quantity for a mixture of strength class C30/37, with a design life of 50 years and an Exposure Class of XC3 is given as 310 kg/m³ (Roadstone, 2019).

For CP114, the cement quantity was calculated using the nominal concrete mix ratio, as well as the density of cement. The density of cement was found in the *British Standard B.S.648:1964 Weights of Building Materials*. The nominal mix ratio was 1:1:2 and therefore consists of 4 parts; one part cement to one part sand to one part coarse aggregates, as in Equation 92.

$$Cement\ Quantity = \frac{1}{4} \times 1m^3 \times \rho_{cem} = 360\ kg/m^3 \qquad \text{(Equation 92)}$$

Where:
- ρ_{cem} is the density of the cement = 1440 kg/m³

Water/cement ratio:
The maximum water/cement (w/c) ratio of 0.55 is specified in the Roadstone guidance document. This ratio was also used in the CP114 mixtures for consistency, and because the water content is not stipulated in the nominal mixtures approach and an assumption was made. The water content was automatically generated by the Embodied Carbon Calculator based on the w/c ratio.

Aggregate quantity:
For the Eurocode mixture, the aggregate quantity was determined by subtracting the masses of the cement and the water, per cubic meter, from the density of concrete. However, in this instance, the density of concrete without reinforcement was used, as shown in Equation 93.

$$Aggregate\ quantity = \frac{2400kg}{m^3} - \frac{310kg}{m^3} - \frac{170.5kg}{m^3} = 1919.5\ kg/m^3 \qquad \text{(Equation 93)}$$

The quantity of sand and coarse aggregate required in the nominal mix from CP114 was calculated in a similar manner as the cement. This is shown in Equations 94 and 95. The density of sand and coarse aggregate was found in the *British Standard B.S.648:1964 Weights of Building Materials*. The total aggregate quantity was 1125.5 kg/m³.

$$Sand\ quantity = \frac{1}{4} \times 1m^3 \times \rho_{fine\ agg} = 400.5\ kg/m^3 \qquad \text{(Equation 94)}$$

$$Coarse\ aggregate\ quantity = \frac{2}{4} \times 1m^3 \times \rho_{coarse\ agg} = 725\ kg/m^3 \qquad \text{(Equation 95)}$$

Where:
- $\rho_{fine\ agg}$ is the density of the sand or fine aggregate = 1602 kg/m³
- $\rho_{coarse\ agg}$ is the density of the coarse aggregate = 1450 kg/m³

Table 32: Concrete embodied carbon software inputs for 1m³ of each mix for the beam and column designs

Input	MIX 1: Eurocodes	MIX 2: CP114 – Mild Steel	MIX 3: CP114 – High Yield Steel
Cement type	CEM I: Portland Cement	CEM I: Portland Cement	CEM I: Portland Cement
Cement quantity	310 kg/m³	360 kg/m³	360 kg/m³
Water/cement ratio	0.55	0.55	0.55
Water content	170.5 kg/m³	198.3 kg/m³	198.3 kg/m³
Aggregate quantity	1919.5 kg/m³	1125.5 kg/m³	1125.5 kg/m³

4.12.2.2. Steel Reinforcement Inputs

The quantity of reinforcing steel required in one cubic meter of each mix was determined using Equations 96. The density of steel reinforcement was found in the *British Standard B.S.648:1964 Weights of Building Materials*. The steel reinforcement quantities are summarised in Tables 33 and 34 for the beams and columns respectively.

$$Steel\ quantity = \frac{A_s \times l \times \rho_s}{V_c} \qquad \text{(Equation 96)}$$

Where:
- A_s is the area of reinforcement provided, as per Tables 28 and 29.
- l is the length of the beam or column, taken as 4m or 3m, respectively.
- ρ_s is the density of the steel reinforcement = 7850 kg/m³
- V_c is the volume of concrete less the reinforcement, as per Tables 30 and 31.

Table 33: Simply supported beam reinforcement quantity inputs to the embodied carbon software

Input	Units	MIX 1: Eurocodes	MIX 2: CP114 – Mild Steel	MIX 3: CP114 – High Yield Steel
Steel quantity in 1m³ of concrete	kg/m³	99.814	186.933	132.621

Table 34: Column reinforcement quantity inputs for the embodied carbon software

Input	Units	MIX 1: Eurocodes	MIX 2: CP114 – Mild Steel	MIX 3: CP114 – High Yield Steel
Steel quantity in 1m³ of concrete	kg/m³	78.100	68.172	70.818

In the Embodied Carbon Calculator, there are three options for the type of steel reinforcement: *World average*, *Europe recycled*, or *Custom EPD*. For this design case, the Custom EPD option was selected. EPD stands for Environmental Product Declarations, and these are created by manufacturers in industry who have completed a Life Cycle Assessment for their products. The Embodied Carbon Calculator uses the quantities from a Life Cycle Assessment in Stages A1 to A3, which is known as Cradle-to-Gate (Circular Ecology, 2019). These LCA stages are discussed in *Section 2.2.4*.

For the Custom EPD option, the tool requires a Global Warming Potential (GWP) quantity. In the absence of an industry EPD that would apply to the current and historic cases, a default value of 1.4 kgCO_2e was used from the ICE database (Hammond & Jones, 2011).

A summary of the concrete mix design inputs to the embodied carbon software is shown in Appendix A, and the reinforcement quantity inputs are shown in Appendix B.

4.12.3. Embodied Carbon Quantities

The results generated by the embodied carbon software for 1 cubic metre of concrete are found in Appendix C. These results are also summarised in Tables 35 and 36, for the beams and columns respectively. The results which were generated by the Embodied Carbon Calculator were for 1m³ of the concrete mix and reinforcement. Therefore, the embodied carbon specific to the designed beams and columns was calculated using Equation 97.

$$Specific\ Embodied\ Carbon = EC_{cube} \times V_c \qquad \text{(Equation 97)}$$

Where:

- EC_{cube} is the embodied carbon per cubic metre of concrete, as calculated by the Embodied Carbon Calculator.
- V_c is the volume of concrete less the reinforcement, as per Tables 30 and 31.

Table 35: Embodied carbon results for the simply supported beams

Output	Units	MIX 1: Eurocodes	MIX 2: CP114 – Mild Steel	MIX 3: CP114 – High Yield Steel
Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	488	602	528
Total Embodied Carbon of the element	kgCO_2e	121.43	222.26	196.27

Table 36: Embodied carbon results for the columns

Output	Units	MIX 1: Eurocodes	MIX 2: CP114 – Mild Steel	MIX 3: CP114 – High Yield Steel
Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	419	440	444
Total Embodied Carbon of the element	kgCO_2e	634.8	1853.0	1801.9

5. RESULTS FOR ADDITIONAL DESIGN CASES

5.1. Overview

Several simply supported beams and columns were designed in order to get a further understanding of the effects the design codes have on the required dimensions and reinforcement area of the elements. Different beam and column lengths, concrete strengths, and applied loads were tested to establish the effects on the cross-sectional dimensions, and ultimately the embodied carbon of each of the designs.

5.2. Simply Supported Beam Designs

5.2.1. Embodied Carbon Calculator Inputs

The results of the 10 simply supported beam designs carried out as part of the research project are shown in Appendix D for the design to the Eurocodes and the Code of Practice CP114. Each design was carried out using the same methodology as discussed in Section 3 and detailed in Sections 4.4 and 4.5. Specific parameters were altered in each of the designs, as detailed below.

Simply supported beam 1:
The calculation procedure for the first simply supported beam designs to the Eurocode and CP114 were described in Sections 4.4 and 4.5, respectively. This first design was used as the baseline design. Therefore, each of the subsequent designs were based on this beam, with one parameter changed in each iteration.

- Concrete strength: C30/37 MPa.
- Beam length: 4000 mm.
- Beam width: 150 mm
- Load: the design load was calculated as per Section 4.4.8 for the Eurocode design and Section 4.5.4 for the CP114 design.

Simply supported beams 2 - 10:
In each of the beam designs, one of the above parameters was altered, as shown in Table 37.

Table 37: Simply Supported Beam Designs 2 - 10 and the altered parameters in each design

Beam	Concrete Strength (MPa)	Length (mm)	Width (mm)	Load (kN)
Beam 2	C30/37	4000	250	Load calculated in Section 4.4.8 for the Eurocodes design and Section 4.5.4 for the CP114 design.
Beam 3	C30/37	5000	250	
Beam 4	C30/37	6000	250	
Beam 5	C30/37	3000	250	
Beam 6	C40/50	4000	250	
Beam 7	C28/35	4000	250	
Beam 8	C30/37	4000	250	Load + 60kN permanent load
Beam 9	C30/37	4000	250	Load + 90kN permanent load
Beam 10	C30/37	4000	250	Load + 120kN permanent load

Embodied Carbon Results

For each of the beams designed to the Eurocodes and CP114, the embodied carbon was calculated using the Embodied Carbon Calculator. The parameters required as inputs to this software were calculated in the same way as in Section 4.12.

Material quantities

The following inputs for each beam design were tabulated and can be found in Appendix E.

- Concrete volume
- Reinforcement volume
- Concrete volume less the reinforcement
- Steel quantity per cubic metre of concrete

Concrete Mixture and Reinforcement

The inputs to the Embodied Carbon Calculator used in Section 4.12.2 for the concrete mixture were used in all the beam designs. These were summarised in Table 32.

Additionally, the same type of reinforcement was used, with a Global Warming Potential (GWP) of 1.4 kgCO_2e.

5.2.2. Embodied Carbon Calculator Results

As described in Section 4.12, the embodied carbon in 1 m^3 of concrete was determined using the Embodied Carbon Calculator. The embodied carbon of the entire element was then calculated using Equation 97. A summary of these results is found in Table 38. The detailed inputs into and the results from the Embodied Carbon Calculator can be sound in Appendix E.

Table 38: Embodied Carbon Results per cubic meter from the Embodied Carbon Calculator and the Total Embodied Carbon per Simply Supported Beam

Beam	Output	Units	MIX 1: Eurocodes	MIX 2: CP114 – Mild Steel	MIX 3: CP114 – High Yield Steel
1	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	488	602	528
	Total Embodied Carbon of the element	kgCO_2e	121.4	222.3	196.3
2	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	438	608	495
	Total Embodied Carbon of the element	kgCO_2e	151.5	314.3	258.9
3	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	464	601	516
	Total Embodied Carbon of the element	kgCO_2e	257.5	513.9	444.3
4	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	453	617	496
	Total Embodied Carbon of the element	kgCO_2e	389.1	812.6	660.7
5	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	399	607	495
	Total Embodied Carbon of the element	kgCO_2e	83.1	164.4	135.5
6	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	424	694	562
	Total Embodied Carbon of the element	kgCO_2e	146.9	309.2	253.4
7	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	415	585	505
	Total Embodied Carbon of the element	kgCO_2e	156.2	297.6	258.8
8	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	467	605	518
	Total Embodied Carbon of the element	kgCO_2e	202.5	407.6	351.8
9	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	458	608	502
	Total Embodied Carbon of the element	kgCO_2e	217.0	451.1	376.1
10	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	440	607	509
	Total Embodied Carbon of the element	kgCO_2e	230.4	491.8	416.2

5.3. Column Designs

5.3.1. Embodied Carbon Calculator Inputs

The results of the 9 column designs carried out as part of this research project are shown in Appendix D for the design to the Eurocodes and the Code of Practice CP114. Each design was carried out using the same methodology as discussed in Section 3 and detailed in Sections 4.9 and 4.10. Specific parameters were altered in each of the designs, as detailed below.

Column 1:
The calculation procedure for the first column design to the Eurocode and CP114 was described in Sections 4.9 and 4.10, respectively. This first design was used as the baseline design. Therefore, each of the subsequent designs were based on this column, with one parameter changed in each iteration.

- Concrete strength: C30/37 MPa.
- Column length: 3000 mm.
- Load: the design load was calculated as per Section 4.9.3 for the Eurocode design and Section 4.10.2 for the CP114 design.

Columns 2 - 9:
In each of the column designs, one of the above parameters was altered, as shown in Table 39.

Table 39: Column Designs 2 - 9 and the altered parameters in each design

Column	Concrete Strength (MPa)	Length (mm)	Load (kN)
Column 2	C30/37	3000	Load calculated in Section 4.9.3. for the Eurocodes design and Section 4.10.2 for the CP114 design
Column 3	C30/37	2000	
Column 4	C30/37	4000	
Column 5	C40/50	3000	
Column 6	C28/35	3000	
Column 7	C30/37	3000	Load + 60kN permanent load
Column 8	C30/37	3000	Load + 90kN permanent load
Column 9	C30/37	3000	Load + 120kN permanent load

Embodied Carbon Results
For each of the columns designed to the Eurocodes and to CP114, the embodied carbon was calculated using the Embodied Carbon Calculator. The parameters required as inputs to this software were calculated in the same way as described in Section 4.12.

Material quantities
The following inputs for each column design were tabulated and can be found in Appendix E.

- Concrete volume
- Reinforcement volume
- Concrete volume less the reinforcement
- Steel quantity per cubic metre of concrete

Concrete Mixture and Reinforcement

The inputs to the Embodied Carbon Calculator used in Section 4.12.2 for the concrete mixture were used in all the column designs. These were summarised in Table 32.

Additionally, the same type of reinforcement was used, with a Global Warming Potential (GWP) of 1.4 kgCO_2e.

5.3.2. Embodied Carbon Calculator Results

As described in Section 4.12, the embodied carbon in 1 m^3 of concrete was determined using the Embodied Carbon Calculator. The embodied carbon of the entire element was then calculated using Equation 97. A summary of these results is found in Table 40. The detailed inputs into and the results from the Embodied Carbon Calculator can be sound in Appendix E.

Table 40: Embodied Carbon Results per cubic meter from the Embodied Carbon Calculator and the Total Embodied Carbon per Column

Column	Output	Units	MIX 1: Eurocodes	MIX 2: CP114 – Mild Steel	MIX 3: CP114 – High Yield Steel
1	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	419	440	444
	Total Embodied Carbon of the element	kgCO_2e	634.8	1855.0	1801.9
2	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	432	440	443
	Total Embodied Carbon of the element	kgCO_2e	622.1	1885.4	1843.6
3	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	461	440	443
	Total Embodied Carbon of the element	kgCO_2e	442.6	1256.9	1229.1
4	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	357	440	443
	Total Embodied Carbon of the element	kgCO_2e	867.5	2513.8	819.4
5	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	390	440	443
	Total Embodied Carbon of the element	kgCO_2e	561.6	1885.4	1843.6
6	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	476	440	443
	Total Embodied Carbon of the element	kgCO_2e	685.4	1885.4	1843.6
7	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	597	503	468
	Total Embodied Carbon of the element	kgCO_2e	947.8	2897.3	2695.7
8	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	616	497	467
	Total Embodied Carbon of the element	kgCO_2e	1073.3	3308.2	3108.5
9	Embodied Carbon in 1m³ of concrete mix	kgCO_2e/m³ concrete	604	478	452
	Total Embodied Carbon of the element	kgCO_2e	1150.3	3641.2	3443.2

6. DISCUSSION

The reinforced concrete elements described in Sections 3, 4 and 5 were designed using two different structural design codes which were in use more than 50 years apart. The elements were designed to the Eurocodes, which are the structural design codes currently in use in Europe. The elements were also designed to the Code of Practice CP114, which was in use during the 1960's. The structural designs for each code differ in terms of their design methodologies, safety factors, and results.

6.1. General Properties and Characteristic Concrete Strength

6.1.1. Concrete

The first major difference between the codes was that the historic Code of Practice CP114 referred to nominal concrete mixes, with a standardised approach to using these concrete mixes in the design. The Eurocodes do not refer to nominal mixes and instead refer only to performance-based characteristics. Therefore, a concrete mix is designed to reach certain strength requirements based on its design life and exposure class making allowance for the durability of the material. This is in contrast to fulfilling certain mix volume criteria, based on the compressive strength.

In the Eurocodes, the process for determining the required characteristic concrete strength is dependent on two factors that were not considered in CP114. These are the design life of the element and the exposure class of the environment in which the element is situated. From these two factors, the code stipulates which concrete class is required. CP114 does not specify a methodology for determining which concrete class or strength should be used.

In the CP114 code, cube strength values were provided, whereas in the Eurocodes, the primary criteria used is cylinder strength. In the Eurocodes, f_{ck} is used to indicate the cylinder strength, which is higher than cube strength. There is a relationship between cube strength and cylinder strength. The compressive strength determined from a concrete cube test is approximately 1.25 more than the compressive strength from a concrete cylinder test. For example, for the C30/37 concrete strength used in the Eurocode designs in this research, the cube strength of 37 MPa is 1.23 times more than the cylinder strength of 30 MPa. Therefore, in the two designs that ran simultaneously, the strength of the concrete used in the Eurocodes design was higher. The characteristic concrete strength specified in each code is highlighted in Table 41.

Table 41: Characteristic Concrete Strength Comparisons

Concrete Strength Property	Units	Eurocode	CP114
Characteristic concrete cube strength	MPa	37	30
Characteristic concrete cylinder strength	MPa	30	-

From the values in Table 41, it is evident that an element designed to the Eurocodes would require a smaller quantity of concrete and reinforcement because the strength of the concrete is higher. Therefore, it can resist greater loads than the concrete used in the CP114 design. For this research, the nominal mix design with the highest cube strength was chosen specifically to close the gap with the strength of the specified Eurocode concrete, for the two to be comparable.

By considering the design life of the structure and the exposure to the environment, it is evident that the Eurocodes have moved towards durability performance which is an important feature in the design of concrete elements. This is an important aspect for the structure as a whole, as a durability-focussed approach typically has less cracking, decreased porosity and corrosion of reinforcement, etc. From a sustainability standpoint, this is also advantageous as there are fewer repairs and materials wasted if a structure is safe and functional throughout its intended design life.

In CP114, the code specifies a certain permissible stress allowed in the concrete element. In the Eurocodes, this is termed the design compressive strength. The values associated with these terms are not the same in each code, but ultimately what they represent is similar. In CP114, the "permissible direct compressive strength" is specified as 7.6 MPa. This is the permissible strength that the member at any cross-section should not surpass. In the Eurocodes, there is a reference to a "Design Compressive Strength". This isn't a flat value that is assigned to the mixture. A partial material factor γ_c of 1.5 is used to determine the design concrete strength, as shown in *Section 3.3.4*. For a C30/37 concrete mix, this translates to a value of 20MPa, which is conservatively less than the 30MPa value of the cylinder strength, whilst still being larger than the permissible stress allowed by the CP114 code, as shown in Table 42. These values differ due to their material safety factors.

Table 42: Permissible and design compressive strength of concrete

Code	Compressive Strength	Units	Value
Eurocodes	Design compressive strength	MPa	20
CP114	Permissible direct compressive stress	MPa	7.6

From these values, it is evident that a beam designed using the Eurocodes can withstand significantly more stress than one designed to CP114. This could be due to the CP114 code being conservative due to unestablished testing methods that have evolved over the past 50 years of design and experimentation. The Eurocodes are less conservative, whilst still being cautious with the use of the partial material safety factor γ_c.

6.1.2. Concrete Density

In Table 43, the concrete density specified by each code is summarised. The Eurocodes account for the density of the steel reinforcement, and therefore the value is higher. However, CP114 uses the density of unreinforced concrete in all the calculations. From this, the Eurocodes are 4% more conservative than CP114.

Table 43: Characteristic density comparison

Property	Units	Eurocodes	CP114
Characteristic Density	kN/m^3	25	24

6.1.3. Steel reinforcement Properties

The properties of steel reinforcement used in concrete have improved since CP114 was the structural design code in use. CP114 refers to two withdrawn British Standards, *B.S.4449, and B.S.4461*, in which there is the option of using mild steel or high-yield steel bars. The differences between the tensile strengths of each type of bar are shown in Table 44. The strength of the high-yield steel reinforcement used in the Eurocodes is also listed in Table 44. The strength of the high-yield steel bars is significantly greater than that of mild steel. This led to less reinforcement required in the elements. This is because the tensile stresses which can be withstood by the element are significantly higher before yielding.

Table 44: Reinforcement yield strength comparison

Reinforcement Property	Units	Eurocodes	CP114: Mild Steel	CP114: High Yield Steel
Tensile Yield Strength	MPa	500	250	410

As with the concrete, the codes discuss the permissible or characteristic tensile strength of the reinforcement, per CP114 and the Eurocodes respectively. These are represented in Table 45. The permissible stress in the high-yield bars is significantly higher than the permissible stress of the mild steel bars.

In the Eurocodes, the material partial factor γ_s of 1.15 was used to calculate the design strength of the reinforcement. However, even whilst using this partial factor, the value used for the reinforcement design strength in the Eurocodes is still significantly higher than the value used for the high-yield bars in CP114. This is because the Eurocodes divide the characteristic strength of the reinforcement by the partial factor of 1.15, whereas CP114 multiplies the characteristic strength by a factor of 0.55, which is more conservative. The evolution of this factor may have been increased due to several reasons, such as improved testing methods and knowledge of material behaviour.

Table 45: Permissible or design tensile strength comparison

Code	Terminology	Unit	Value
Eurocodes	Design tensile strength	MPa	434.78
CP114	Permissible tensile stress in mild steel bars	MPa	140
CP114	Permissible tensile stress in deformed high-yield steel bars	MPa	225.5

6.2. Loading

6.2.1. Dead or Permanent Loads

CP114 refers to the consistent loads acting on a structure as dead loads, whereas the Eurocodes refer to them as Permanent Loads. These are the same type of loads and were calculated in the same manner for both designs. The concrete density in the Eurocodes is 4% higher, therefore, the calculation of subsequent loading is higher, as shown in Tables 46 and 47. The results of Beams 8 – 10 and Columns 7 – 9 are not shown here as additional permanent loads were intentionally added to the calculated load.

On average, the beams designed to the Eurocodes were 2.6% higher than CP114, with the exception of Beam 5, which had the shortest length. However, in the column designs, the permanent loading remained consistent throughout because the dimensions of the supported beams and slabs were not changed in each iteration. The permanent load from the Eurocodes was 4% larger than that of CP114 dues to the larger density prescribed by the Eurocodes.

Table 46: Permanent load comparison in the simply supported beam designs

Beam	Permanent Loads (kN)		Difference (%)
	Eurocodes	CP114	
1	69	67	2.9
2	75	73.2	2.4
3	112.5	109.5	2.7
4	157.5	153	2.9
5	45	44.1	0.9
6	75	73.2	2.4
7	75	73.2	2.4

Table 47: Permanent Load Comparison in the Column Designs

Column	Permanent Loads (kN/m)		Difference (%)
	Eurocodes	CP114	
1 – 6	131.3	126	4

6.2.2. Live or Imposed Loads

CP114 refers to variable loads as live loads, whereas the Eurocodes refer to them as imposed loads. These are the same type of load. For an element situated in an office block, the temporary loads in Table 48 were specified by each respective code. The loading in the Eurocode is slightly more conservative when considering the loads of the beam design, however, the distributed loading used in the column design is the same in both codes.

Table 48: Variable load comparison

Element	Units	Variable Loads		Difference (%)
		Eurocodes	CP114	
Beams	kN	3	2.7	10.5
Columns	kN/m	12.5	12.5	0

6.2.3. Unfactored Loads and Factored Loads

The overall unfactored load applied to the beam was the sum of the permanent and variable loads in Tables 46 and 48. The unfactored loads were more conservative for the Eurocode, as shown in Table 49. The same is true for the overall unfactored loading of the column design, as shown in Table 50. The unfactored Eurocode loading was consistently between 2.5% and 3.7% larger than the unfactored CP114 loading.

Table 49: Unfactored load comparison for the simply supported beam designs

Beam	Unfactored Loads (kN)		Difference (%)
	Eurocodes	CP114	
1	72	69.7	3.2
2	78	75.9	2.7
3	115.5	112.2	2.9
4	160.5	155.7	3.0
5	48.0	46.8	2.5
6	78	75.9	2.7
7	78	75.9	2.7

Table 50: Unfactored load comparison for the column designs

Column	Unfactored Loads (kN/m)		Difference (%)
	Eurocodes	CP114	
1 – 6	143.75	138.5	3.7

From *Sections 3.3.6 to 3.3.8*, it was found that the Eurocodes make use of a more rigorous method in determining the design load. As per CP114, the load factor method makes use of a flat safety factor of 1.8 to determine the overall loading applied to the beam. However, in the Eurocodes, the loading is broken down into several scenarios, and different safety factors are applied to the permanent load, the imposed load, and the wind load. In this design case, the wind load was ignored because the element was situated indoors. This was an iterative process through several favourable and unfavourable conditions, in the ultimate limit state and the serviceability limit state to determine the maximum loading to be considered as acting on the beam. In Tables 51 and 52, the final factored design loads are shown for the simply supported beams and columns, respectively.

Table 51: Factored load comparison for the simply supported beams

Beam	Factored Design Loads (kN)		Difference (%)
	Eurocodes	CP114	
1	97.7	125.4	24.8
2	105.8	136.6	25.4
3	156.4	202.0	25.4
4	217.1	280.3	25.4
5	65.3	84.2	25.3
6	105.8	136.6	25.4
7	105.8	136.6	25.4

Table 52: Factored load comparison for the columns

Column	Factored Design Loads (kN/m)		Difference (%)
	Eurocodes	CP114	
1 – 6	195.9	249.3	23.9

From Table 51, we see that regardless that the unfactored loads from CP114 were less than those calculated for the Eurocodes, the factored load from CP114 was significantly higher than the factored load from the Eurocodes. This holds for the beam and the column design loads.

In the beam designs, the factored design load was approximately 25% higher from the CP114 codes, than from the Eurocodes. The factored design load was 23.9% lower for the columns designed to the Eurocodes than those designed to CP114.

This shows how conservative the load factor method was, ultimately leading to an overdesigned section.

6.3. Simply Supported Beam: Design Comparisons

6.3.1. Deflection

The value used for calculating the optimum length over depth ratio (slenderness ratio) for the beam was the only factor across the codes which was found to be consistent across both. A maximum slenderness ratio of 20 was stipulated in both structural design codes.

6.3.2. Reaction Forces and Moments

The load that was shown previously to act on the beam was significantly different for each of the codes. This difference also translated into the reaction forces at the supports and the maximum bending moment which the beam was designed to resist. The load for the CP114 code was approximately 25% higher than that of the Eurocodes, therefore the forces and moments per the CP114 code were larger as well. This load was higher for the CP114 code due to the large load factor applied to the load. The reaction forces and bending moments for the design of Beam 1 are shown in Table 53 to illustrate the differences in these parameters between the two codes. The beam designed to CP114 would need to be designed to accommodate a much larger bending moment than the beam designed to the Eurocodes.

Table 53: Reaction forces and moment comparison

Parameter	Units	Eurocode Design	CP114 Design
Reaction forces	kN	48.8	62.7
Ultimate bending moment	kNm	97.65	125.4

6.3.3. Resistance Moments and Minimum Dimensions

The maximum moment that an element's cross-section can withstand is referred to as its moment of resistance. It follows that the resistance moment should be more than the ultimate moment expected to act on the beam. The resistance moments are a function of the dimensions of the cross-section of an element and the quantity of reinforcement and therefore, an iterative process was used to determine the optimum dimensions of the beam. The larger the elements cross-section, the higher its resistance moment. Therefore, the height of the beam was continually increased until the moment of resistance was larger than the maximum moment applied to the beam.

The design moments and the resistance moments for each of the beam designs are shown in Table 54. For a set beam width, the heights of the beams were iteratively determined to ensure that the resistance moment was larger than the design moment.

The resistance moments from CP114 are larger than those from the Eurocodes because of the larger dimensions that are required. However, the resistance moment from the Eurocodes are less because the beams can still accommodate the design moment even with the smaller cross-section.

Table 54: Resistance Moments for each Simply Supported Beam Design Case

Beam	Parameter	Units	Eurocode Design	CP114: Mild Steel Design
1	Design Moment M_{max}	kNm	97.7	125.4
	Resistance Moment M_r	kNm	109.7	126.2
2	Design Moment M_{max}	kNm	105.8	136.6
	Resistance Moment M_r	kNm	115.6	138.1
3	Design Moment M_{max}	kNm	195.5	252.5
	Resistance Moment M_r	kNm	229.9	256.0
4	Design Moment M_{max}	kNm	325.7	420.4
	Resistance Moment M_r	kNm	368.5	430.6
5	Design Moment M_{max}	kNm	48.9	63.2
	Resistance Moment M_r	kNm	52	64.0
6	Design Moment M_{max}	kNm	105.8	136.6
	Resistance Moment M_r	kNm	108.1	141.0
7	Design Moment M_{max}	kNm	105.8	136.6
	Resistance Moment M_r	kNm	115.4	136.7
8	Design Moment M_{max}	kNm	186.8	244.6
	Resistance Moment M_r	kNm	223.9	248.1
9	Design Moment M_{max}	kNm	227.3	298.6
	Resistance Moment M_r	kNm	246.4	306.3
10	Design Moment M_{max}	kNm	267.8	352.6
	Resistance Moment M_r	kNm	278.4	361.0

The resistance moments in Table 54 relate to the optimised dimensions which can maintain the load and the resultant moment applied to the beam. These optimised dimensions are shown in Table 55. As seen in this table, the cross-sectional dimensions were less for the Eurocode design than for either of the CP114 designs, in all of the beam design iterations. This correlates to less concrete required in the element and therefore less embodied carbon. This held true regardless of the span, concrete strength, and loading applied to the beams.

The cross-sectional dimensions for the CP114 design cases remained the same whether the mild steel reinforcement or the high-yield steel reinforcement was used. It can therefore be inferred that the CP114 design code did not take sufficient measure to optimise the dimensions of the elements if steel with a higher yield strength was used.

Table 55: Simply Supported Beam Dimensions for each Design Case

Beam	Parameter	Units	Eurocode Design	CP114: Mild Steel Design	CP114: High Yield Steel Design
1	Length	mm	4000	4000	4000
	Breadth	mm	150	150	150
	Height	mm	420	630	630
2	Length	mm	4000	4000	4000
	Breadth	mm	250	250	250
	Height	mm	350	530	530
3	Length	mm	5000	5000	5000
	Breadth	mm	250	250	250
	Height	mm	450	700	700
4	Length	mm	6000	6000	6000
	Breadth	mm	250	250	250
	Height	mm	580	900	900
5	Length	mm	3000	3000	3000
	Breadth	mm	250	250	250
	Height	mm	280	370	370
6	Length	mm	4000	4000	4000
	Breadth	mm	250	250	250
	Height	mm	350	460	460
7	Length	mm	4000	4000	4000
	Breadth	mm	250	250	250
	Height	mm	380	520	520
8	Length	mm	4000	4000	4000
	Breadth	mm	250	250	250
	Height	mm	440	690	690
9	Length	mm	4000	4000	4000
	Breadth	mm	250	250	250
	Height	mm	480	760	760
10	Length	mm	4000	4000	4000
	Breadth	mm	250	250	250
	Height	mm	530	830	830

6.3.4. Required Reinforcement Area

The use of reinforcing steel in reinforced concrete comes at a high cost to the project as it is expensive, and if it is not properly laid and protected, it can have negative effects on the durability of the structure due to rusting. This effect on the durability of the structure can greatly impact the service life of the structure. It is also costly to the environment as greenhouse gases are released during the production of steel. If the quantity of reinforcing steel is significantly reduced, then the cost of the structure and the embodied carbon of the structure would both be greatly reduced.

The cross-sectional area of the reinforcement provided in each design case is shown in Figure 23 for the simply supported beam designs, and Figure 24 for the column designs. These charts are based on the required reinforcement areas provided in Appendix D.

In both the simply supported beam designs and the column designs, the required reinforcement was consistently higher for the CP114 designs compared to the Eurocodes, and higher for if mild steel reinforcement was used. The reinforcement plays a significant role in the embodied carbon of a reinforced concrete element, and therefore these reinforcement quantities are important.

From Figure 23, it is evident that the most reinforcement is required in the beams with longer spans compared to those with lesser spans. Thereafter, more reinforcement was required if the loading on the beam increased. The relative increase in provided reinforcement area is summarised below.

- Eurocodes vs CP114 (mild steel): there was a percentage increase of 88% to 120% in the CP114 design (using mild steel) compared to the Eurocodes.
- Eurocodes vs CP114 (high yield steel): there was a percentage increase of 48% to 85% in the CP114 design (using mild steel) compared to the Eurocodes.
- CP114 (high yield steel) vs CP114 (mild steel): there was a percentage increase of 33% to 57% in the high yield steel design compared to the mild steel design.

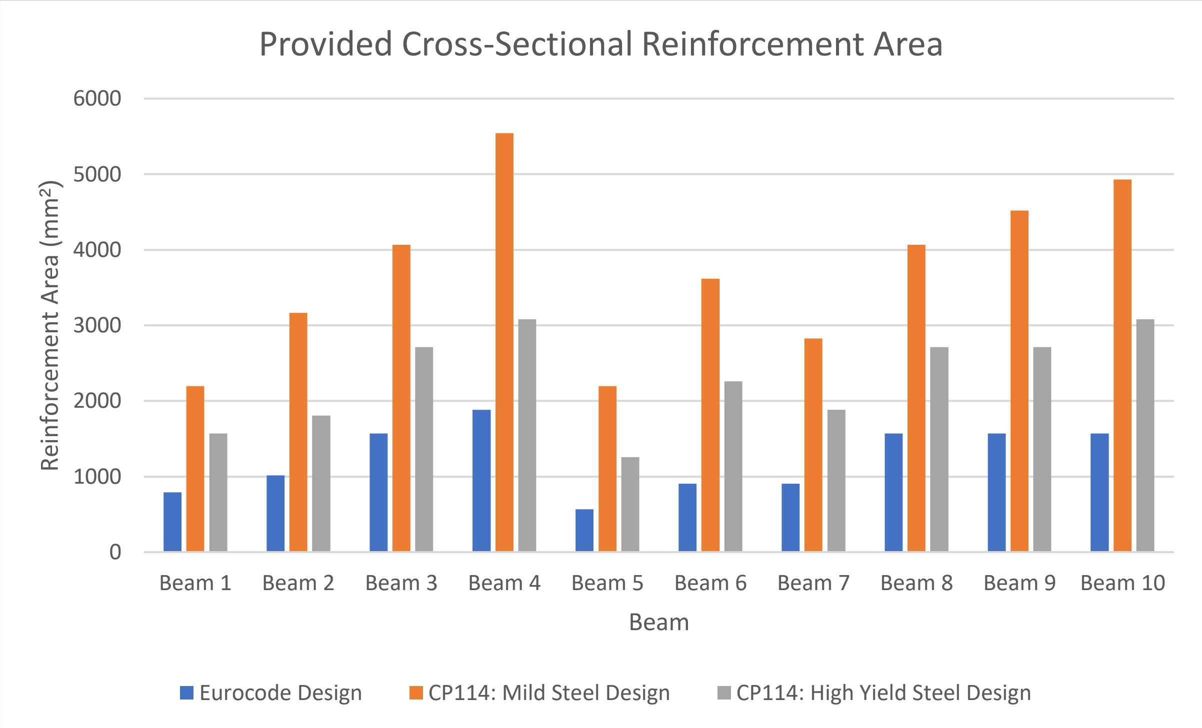

Figure 23:Cross-sectional reinforcement area provided in each simply supported beam design

6.3.5. Design Stresses vs Permissible Stresses

It is important to ensure that the design stresses in a section of the beam can be accommodated to ensure that the beam does not fail. This is done by ensuring that the design stresses are less than the permissible stresses in the concrete and reinforcement using Equation 98. The results for the concrete stresses and the reinforcement stresses are shown in Tables 56 and 57, respectively, for the first beam and column designs. This design was chosen to showcase these parameters. It is evident that the permissible stresses from the Eurocode are significantly higher than those from the CP114 code, once again reiterating that the CP114 codes were conservative in the absence of adequate testing methods and design theories.

$$\sigma = \frac{P}{A} \quad \text{(Equation 98)}$$

Where:

- σ is the stress of the concrete or reinforcement under consideration
- P is the maximum load as shown in Table 51.
- A is the cross-sectional area of the concrete beam or the reinforcement provided.

Table 56: Concrete design versus permissible stress

Code	Units	Design Stress σ	Permissible Stress
Eurocodes	MPa	1.55	20
CP114 – Mild Steel	MPa	1.33	10
CP114 – High Yield Steel	MPa	1.33	10

Table 57: Reinforcement design versus permissible stress

Code	Units	Design Stress σ	Permissible Stress
Eurocodes	MPa	96.02	434.8
CP114 – Mild Steel	MPa	57.05	140
CP114 – High Yield Steel	MPa	79.86	225.5

6.3.6. Concrete Volume

The quantities of concrete required for each of the designs are shown in Appendix E for the simply supported beams. These are shown graphically in Figure 24. Once again, it can be seen that the required concrete for the elements designed to the Eurocodes was less than those designed to CP114.

The results between the CP114 mild steel and high yield steel designs were very similar. It was established that the concrete elements would have the same dimensions regardless of which type of reinforcing steel was used. Only the amount of reinforcement differs in each of the designs.

There was 25-44% more concrete required to sustain the load in the CP114 designs compared with the Eurocode design.

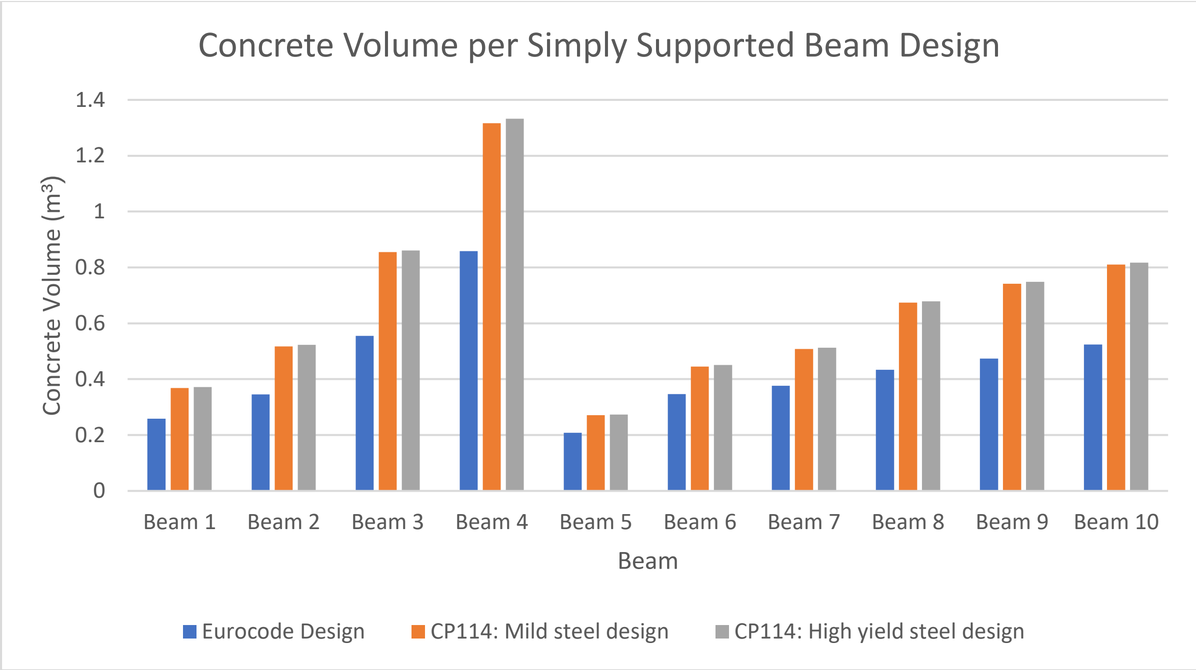

Figure 24: Concrete volume for each simply supported beam design

6.4. Column: Design Comparisons

6.4.1. Slenderness and Effective Height Comparisons

One of the most noticeable differences between the two codes was the slenderness limit. The CP114 codes prescribed a specific value of 15 as the slenderness limit. If the span over depth ratio is less than 15, then the column is short. Otherwise, it is considered long. The Eurocodes, on the other hand, introduced an equation for determining the slenderness limit, which is a function of the concrete strength, the cross-section dimensions, and the axial load applied to the beam. For this design case, the limit was significantly higher than 15. This shows that the CP114 code was conservative in prescribing the limit of 15 for the worst-case design scenario. However, with the inclusion of the slenderness limit equation, the Eurocodes take a more rational design approach. The larger slenderness limit also allows for optimised cross-section dimensions.

Furthermore, the slenderness ratio is calculated differently for the two codes. The CP114 code calculates the slenderness ratio purely as a function of the effective length of the beam. Whereas, the Eurocodes relate the slenderness limit to the effective height as well as the radius of gyration, which is dependent on the effective stiffness between the beams and the columns in the system.

6.4.2. Material Quantities

6.4.2.1. Reinforcement

The cross-sectional area of the reinforcement provided in each design case is shown in Figure 25 for the column designs. These charts are based on the required reinforcement areas provided in Appendix D.

The required reinforcement was consistently higher for the CP114 designs compared to the Eurocodes, and higher for if mild steel reinforcement was used. The reinforcement plays a significant role in the embodied carbon of a reinforced concrete element, and therefore these reinforcement quantities are important.

There is more variability in the required reinforcement for the Eurocode designs due to the introduction of the column reinforcement charts shown in Figure 25. This ultimately leads to optimised designs as the column designer does not need to over-estimate the required reinforcement, which is what happened in the CP114 designs. In the CP114 designs, there is a specified minimum and maximum reinforcement percentage, however, no equation or chart was provided to more accurately determine how much reinforcement is required.

The amount of reinforcement required in the colums was the same regardless of whether mild steel or high yield steel was used for all iterations, except for the increased loading iterations.

The relative increase in provided reinforcement area is summarised below.

- Eurocodes vs CP114 (mild steel): there was a percentage increase of 51% to 131% in the CP114 design (using mild steel) compared to the Eurocodes.
- Eurocodes vs CP114 (high yield steel): there was a percentage increase of 36% to 85% in the CP114 design (using mild steel) compared to the Eurocodes.

- CP114 (high yield steel) vs CP114 (mild steel): there was no difference between the two designs in terms of required reinforcement until additional loading was introduced. Thereafter, there was a percentage increase of 22% to 25% in the mild steel design compared to the high yield steel design.

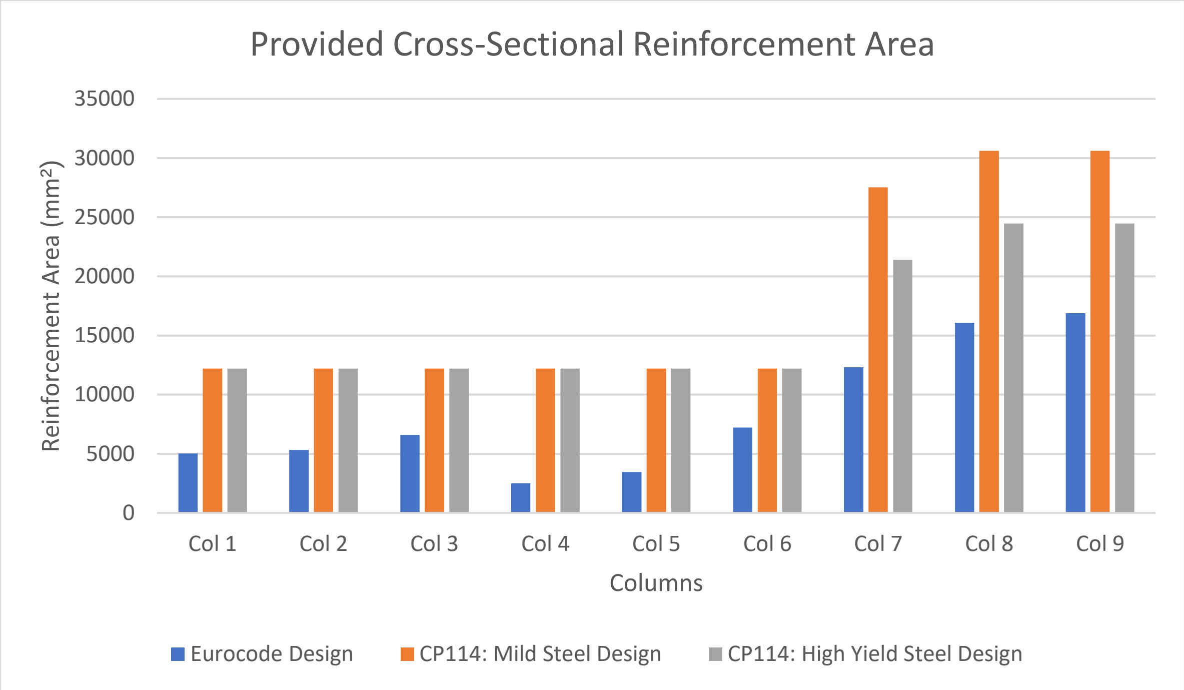

Figure 25: Cross-sectional reinforcement area provided per column design

6.4.2.2. Concrete

The quantities of concrete required for each of the designs are shown in Appendix E, and are shown graphically in Figure 26. It can be seen that the required concrete for the elements designed to the Eurocodes was consistently less than those designed to CP114.

The results between the CP114 mild steel and high yield steel designs were very similar. It was established that the concrete elements would have the similar dimensions regardless of which type of reinforcing steel was used. However, the amount of reinforcement differs in each of the designs.

There was 80-120% more concrete required to sustain the load in the CP114 designs compared with the Eurocode design.

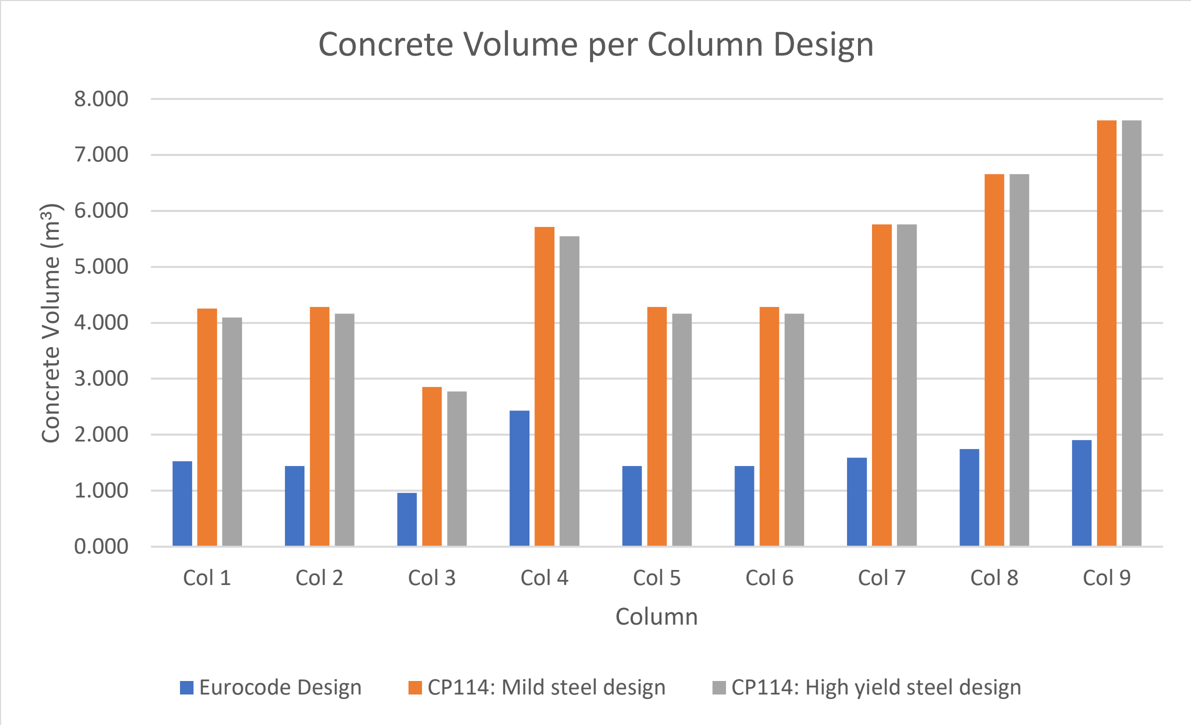

Figure 26: Concrete volume for each column design

6.5. Embodied Carbon

The results generated by the Embodied Carbon Calculator for 1 cubic metre of concrete are shown for the first simply supported beam and column designs in Appendix C. The embodied carbon results for the rest of the beam and column designs are also summarised in Appendix E.

The inputs for the concrete mix designs used for the Eurocodes and the CP114 codes can be found in Table 32. For each cubic meter of concrete produced, the nominal mixture from CP114 required 50.5 kg more cement, as well as 27.8kg more water. The concrete mixture from the CP114 design required 794kg more aggregate than the Eurocode's concrete mixture. The above differences resulted in a larger quantity of embodied carbon from the CP114 design, compared to the Eurocodes design. Additionally, there was consistently more steel required in a cubic meter of concrete for the CP114 designs than for the Eurocode designs.

However, the two main factors which result in high embodied carbon of the elements is the volume of concrete and the volume of reinforcement required to sustain the loads. A reduction in the volume of concrete required in an element reduces the embodied carbon of the element. Similarly, a reduction in the required reinforcement also reduces the embodied carbon of the element.

The Embodied Carbon Calculator determines the embodied carbon from 1m³ of concrete. The embodied carbon results per cubic meter of concrete were then multiplied by the total volume of concrete in the element to determine the embodied carbon of the entire element. These results are tabulated and summarised in Tables 38 and 40, for the simply supported beams and columns, respectively. These are also summarised in Appendix E.

The embodied carbon results for the entire elements are shown graphically below in Figure 27 for the simply supported beam designs and Figure 28 for the column designs.

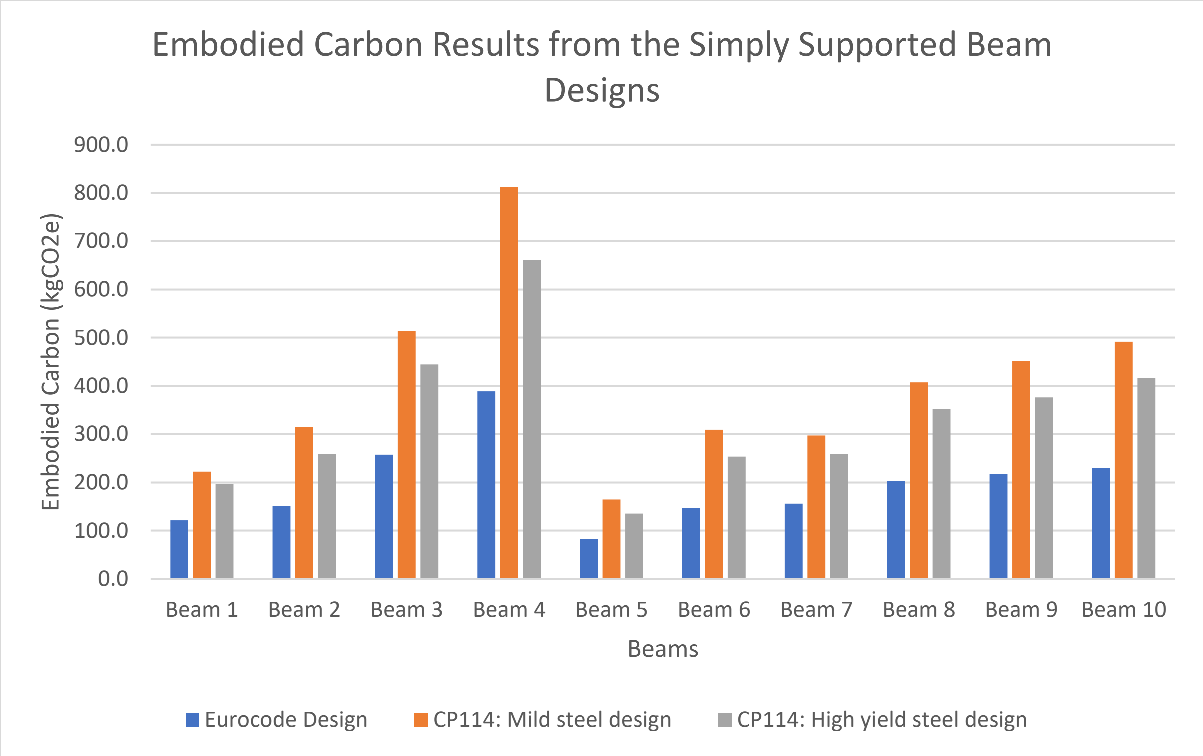

Figure 27: Embodied carbon results from the simply supported beam designs

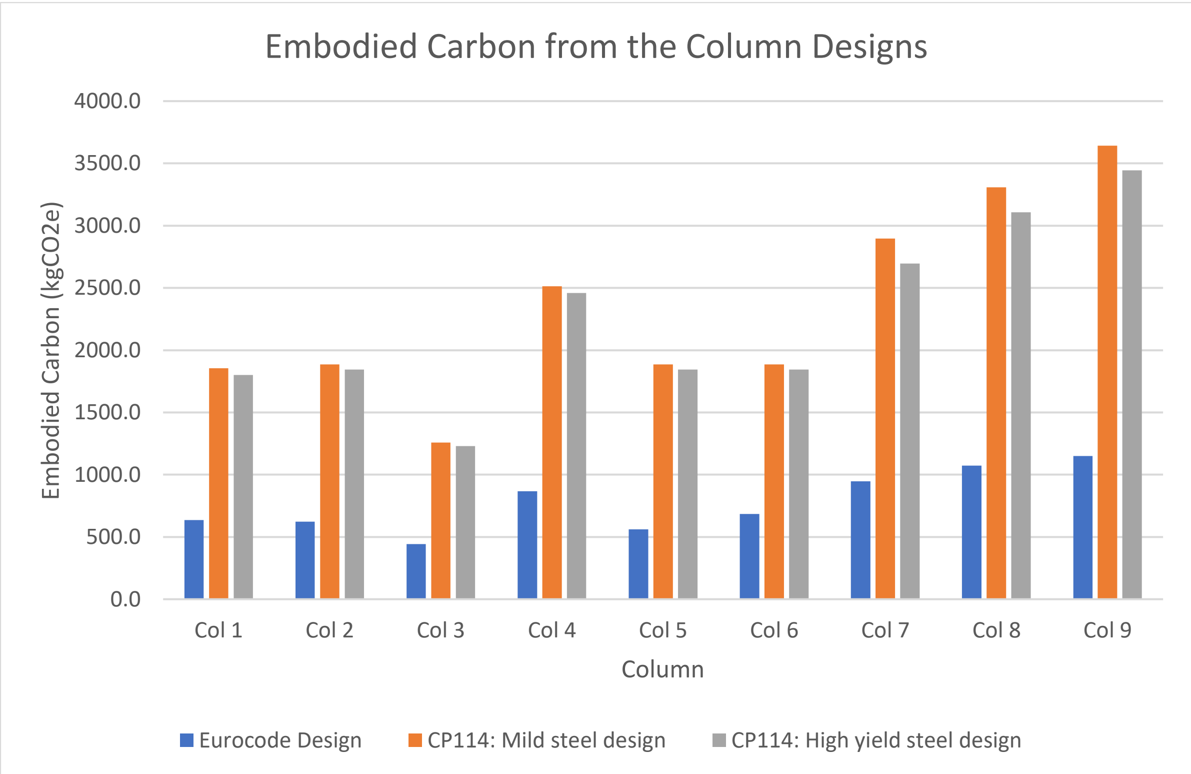

Figure 28: Embodied carbon results from the column designs

From the graphs shown in Figures 27 and 28, and the tabulated embodied carbon results in Tables 38 and 40, it is evident that the embodied carbon is less for the structural design of the Eurocodes, for both the simply supported beam design and the column designs. Additionally, the embodied carbon of the CP114 structural designs was greater if mild steel was used, rather than high-yield steel. This difference was more pronounced for the simply supported beam than for the column. Additionally, this difference is based primarily on the significant amount of reinforcement required by the CP114 code compared to the Eurocodes.

To determine the average embodied carbon savings as a percentage between the Eurocodes and the CP114 code, Tables 58 and 59 were developed. The results are outlined below.

Simply Supported Beams

The embodied carbon of the Eurocodes designs was on average 67% less than the CP114 design if mild steel was used, and 52% less if high yield steel was used. This percentage was slightly higher for the iterations with longer spans and higher loads. Therefore, it can be inferred that the gap between the embodied carbon for each of the codes would get bigger if the spans or the loads continued to increase.

The embodied carbon difference between the mild steel and high yield steel designs to CP114 was less pronounced, at an average of 17%. The primary difference between these two designs was the quantity of reinforcement required to sustain the load. Whereas the amount of concrete required in each design was very similar.

Table 58: Comparison of Embodied Carbon Results for the Simply Supported Beam Designs

Comparison of Embodied Carbon Results for the Simply Supported Beam Designs			
Beam	**Eurocode vs CP114 (mild steel)**	**Eurocode vs CP114 (high yield steel)**	**CP114 (mild steel) vs CP114 (high yield steel)**
Beam 1	59%	47%	12%
Beam 2	70%	52%	19%
Beam 3	66%	53%	15%
Beam 4	70%	52%	21%
Beam 5	66%	48%	19%
Beam 6	71%	53%	20%
Beam 7	62%	49%	14%
Beam 8	67%	54%	15%
Beam 9	70%	54%	18%
Beam 10	72%	57%	17%
Average	**67%**	**52%**	**17%**

Columns

The embodied carbon of the Eurocodes designs was on average 100% less than the CP114 design if mild steel was used, and 97% less if high yield steel was used. This percentage was slightly higher for the iterations with longer lengths and higher loads.

The embodied carbon difference between the mild steel and high yield steel designs to CP114 was less pronounced than that of the beams, at an average of 4%.

Table 59: Comparison of Embodied Carbon Results for the Column Designs

Comparison of Embodied Carbon Results for the Column Designs			
Column	**Eurocode vs CP114 (mild steel)**	**Eurocode vs CP114 (high yield steel)**	**CP114 (mild steel) vs CP114 (high yield steel)**
Column 1	98%	96%	3%
Column 2	101%	99%	2%
Column 3	96%	94%	2%
Column 4	97%	96%	2%
Column 5	108%	107%	2%
Column 6	93%	92%	2%
Column 7	101%	96%	7%
Column 8	102%	97%	6%
Column 9	104%	100%	6%
Average	**100%**	**97%**	**4%**

As discussed above, there was an unintentional reduction in embodied carbon when comparing the Eurocode designs and the CP114 designs. This stems from the evolution of structural testing methods and increased knowledge about material properties and design philosophies.

6.6. Summary

The Eurocodes are based on the Limit State Design (LSD) methodology, whereas the CP114 code is based on the Allowable Stress Design (ASD) methodology.

From Sections 6.3 to 6.5, it is evident that the LSD method is more economical than the ASD method as the cross-sectional area of the concrete was optimised, which resulted in smaller volume of required concrete. This is shown in the graphs in Figures 24 and 26 for the beam and column designs respectively.

Importantly, less concrete does not automatically translate to lower embodied carbon because reinforced concrete comprises two very different materials which perform different functions. If the amount of reinforcement is increased significantly to balance the reduced dimensions of the concrete, then the embodied carbon will increase as well.

As is shown in this research project, the amount of reinforcement differs greatly depending on which code is used. However, in Figures 23 and 25, it is evident that the required steel reinforcement is significantly less in the Eurocode designs than the CP114 designs. This is because the Eurocodes are based on the LSD methodology, which uses the updated understanding of the actual relationship between concrete and reinforcing steel, determined through statistical probabilistic methods. The ASD method used by CP114 is based on the outdated linear elastic theory for reinforced concrete which assumes that the element will fail when the elastic limit is reached, as shown in Figure 7. Therefore, more steel is required in this design in order to ensure that the elastic limit is not reached.

However, when high yield steel was used in the CP114 design, then the volume of reinforcement required was less than when mild steel reinforcement was used. This is because the forces which can be sustained by the high yield steel were significantly higher. Therefore, on average 17% less reinforcement was required in beam designs, and 4% less in columns.

A major difference between the codes was how the ultimate load was determined, and the safety factors used. CP114 used only one safety factor which was applied to the total load. Whereas this is an easier calculation process than the Eurocodes, it did lead to significantly higher loads which needed to be sustained by the elements. This could be one of the primary factors in why more concrete and reinforcement was required in the elements design to the CP114 codes.

On the other hand, the Eurocodes make use of a robust method of determining the ultimate design load. This method is complicated and takes significantly more time to determine, but results in a lower and more realistic load which needs to be resisted by the elements.

The primary differences between the codes are summarised below:

- The Eurocodes specified a higher concrete strength because the cylinder strength of concrete was considered. Therefore, while the Eurocodes specify a C30/37 concrete, which has a cibe strength of 37MPa, the corresponding mix in the CP114 code is a concrete strength of 30MPa.
- The concrete density used in the Eurocodes is higher than that used in the CP114 code because the reinforcement is taken into consideration. Due to this, the loadings calculated for the Eurocodes were higher than those calculated by the CP114 code.
- Due to the higher loadings discussed in the point above, it would be assumed that the Eurocodes were more conservative in terms of the design load. However, because the CP114 code merely applied one flat safety factor to the load, the factored design load used in the CP114 design was significantly higher than that from the Eurocodes. This is because the Eurocodes use partial safety factors which are applied to their corresponding load type.
- The reinforcement strength used in designs today is 500MPa, whereas the reinforcement which was in use 50 years ago was between 250MPa and 410MPa. This resulted in a larger volume of reinforcement required as per the historic code. The use of reinforcing steel in reinforced concrete comes at a high cost to the project as it is expensive, and if it is not properly laid and protected, it can have negative effects on the durability of the structure due to rusting. This effect on the durability of the structure can greatly impact the service life of the structure. It is also costly to the environment as greenhouse gases are released during the production of steel. If the quantity of reinforcing steel is significantly reduced, then the cost of the structure and the embodied carbon of the structure would both be greatly reduced.
- As part of the Eurocodes, the column reinforcement charts, as shown in Figure 15, were developed to determine the cross-sectional area of reinforcement required. However, in the CP114 code, there is no chart or equation used to determine the required reinforcement. A minimum and a maximum reinforcement area are specified, but this is a large range, and the amount of reinforcement used is set at the designer's discretion. This leads to an over reinforced section in order to ensure that the column does not fail.
- The slenderness limits for the column designs were different between the codes. Whereas the CP114 code specified a set, and relatively low, limit, the Eurocodes make use of a changing slenderness limit, which takes design-specific factors into account. This prevents the code from becoming too restrictive.

7. CONCLUSION

7.1. Overview

Reinforced concrete is a widely used engineered material due to its many advantages, such as strength, versatility, workability, and durability. However, the production of cement and reinforcing steel emit a significant quantity of carbon dioxide and other greenhouse gases into the atmosphere. This corresponds to a high embodied carbon for reinforced concrete structures. Embodied carbon is the amount of greenhouse gases emitted during all stages of a product's life, including extraction, transport, manufacture, and installation (EPA, 2023)

The 2022 Global Status Report for Buildings and Construction found that the construction sector accounts for approximately 37% of energy related carbon dioxide emissions into the atmosphere (UNEP, 2022). Many of the Sustainable Development Goals (SDGs), as described in Section 2.2.2, are linked directly to the construction sector. This is further motivation and reason for the construction sector to innovate and strive for decarbonisation and improving the overall sustainability of the industry.

Goals 12 and 13, which call for the responsible consumption and use of materials and for climate action respectively, relate specifically to the scope of this research project. Accurately designed reinforced concrete elements require a smaller volume of materials to perform their given tasks. This reduction in the quantity of required materials assists directly in meeting the targets of Goal 12. Subsequently, this reduces the embodied carbon dioxide emissions from the structure by eliminating the use of unnecessary materials, which is a target of Goal 13.

This study was used to determine the carbon dioxide savings purely from the reduction of a reinforced concrete element’s dimensions as well as its material quantities, without considering the quality and composition of those materials. Therefore, SCMs were not introduced into the scope of this research. Only CEM I cement was used in the concrete mix design.

According to P.K. Mehta, one of the primary methods for reducing emissions from the construction sector is by reducing the quantities of materials used and wasted. In concrete, this would translate into the reduction of concrete volumes and reinforcement quantities. A reduction in the materials required for an element to perform its intended function leads to a reduction in the embodied carbon of the element.

A reduction in the volume of concrete required in an element reduces the embodied carbon of the element. Similarly, a reduction in the required reinforcement also reduces the embodied carbon of the element.

There have been several evolutions to the structural building codes over the past decades, with varying structural design methodologies, from the Allowable Stress Design (ASD) method to the Limit State Design (LSD) method. Each had a unique design philosophy and associated safety factors. The unintentional effects that the evolution of the design codes has had on the sustainability associated with the structural elements were investigated in this research report.

The beams and columns were designed using the current and the historic structural design codes. Different reinforced concrete sections were generated from each of the codes due to the differences in the analysis procedures specified in each of them. Therefore, using a similar assumed embodied carbon for each of the concrete design mixes, any changes in the carbon dioxide emissions were derived directly from the optimization of the dimensions of the section of the member, and the required reinforcement to sustain the load.

Two structural design codes were investigated. These codes were in use more than 50 years apart. The Eurocodes are the structural design codes currently used in the European Union, whereas the British Standards Code of Practice CP114 was in use in the 1960s and 1970s. The Eurocodes use the LSD methodology whereas CP114 refers to the ASM method.

Two structural building elements were designed to each of these codes; namely, a simply supported beam and a column. These two elements were chosen as they are common building elements, and therefore a reduction of the embodied carbon of these members is particularly important and effective. They also represent a bending member and a compression member, and the results for each type of member could be compared.

Across the designs of both elements, there was a decrease in the quantity of concrete and reinforcement required to sustain the design loads in the Eurocodes compared to CP114. The reduction in the volumes of concrete and reinforcement corresponds to a reduction in the element's embodied carbon.

Using an Embodied Carbon Calculator software, the embodied carbon of each of the beams and columns was determined when they were designed to the Eurocodes and to the CP114 codes.

CP114 refers to mild steel reinforcement as well as high-yield steel reinforcement. The beams and columns were designed using mild steel and high yield steel as part of the CP114 code in order to compare the results. Whereas there were large differences in the embodied carbon when comparing the Eurocode designs with the CP114 designs, the comparison between the mild steel design and the high yield steel design were less pronounced. For the simply supported beams, there was a 17% reduction in embodied carbon from the use of high yield steel. However, for the columns, the difference was only 4%.

On average, the beams designed to the Eurocode resulted in 67% less embodied carbon than beams designed to the CP114 code if mild steel reinforcement was used. When high yield reinforcement was used in the CP114 design, the embodied carbon was 52% less, on average.

The column designed to the Eurocodes had on average 97-100% less embodied carbon emissions compared to the columns designed to CP114, depending on the type of steel reinforcement used.

The primary reason for these differences in the required concrete and reinforcement quantities, and ultimately the embodied carbon, is due to the design methodology on which each code is based. In the CP114 design methodology, based on the ASD methodology, the permissible stresses in the concrete and reinforcement are specified, and should not be exceeded. However, these limits are based on the assumed linear elastic behaviour of reinforced concrete, which is an outdated assumption, and in modern times, non-linear stress-strain curves of concrete and reinforcement are used. This non-linear assumption allows for materials to be subject to higher stresses in the concrete and reinforcement. Therefore, the permissible stresses are conservative and lead to unnecessarily large quantities of materials required for a given structural element. The Eurocodes do not specify set permissible stresses for concrete and reinforcement.

A reason for the improved environmental impact of the design code evolution is that, through research and development over time, design codes are able to account for the more efficient use of materials, less variability in material performance, and the improved quality control of construction methodologies. This points to the importance of continued research on concrete materials and their use for improved sustainability.

The Eurocodes are based on the LSD methodology, which analyses a structural element in terms of its ultimate limit state or its sustainability limit state. The adoption of this thinking has improved the durability of a structure, as it is designed with consideration to its intended service life and environmental exposure, such as proximity to the ocean, humidity, snow, etc. The improved durability of the structure ensures that the structures will reach its intended service life.

Additionally, the LSD method adopts two partial factors which are applied to the loads and the material strength, whereas the ASD method only has one safety factor which is applied to the loading. This safety factor is high, and therefore the loads that are designed for in CP114 are subsequently larger than the Eurocodes. This leads to an over-designed and possibly over-reinforced structure or element.

Specific parameters were changed and tested as part of the research project. However, there was consistently a reduction in embodied carbon from the Eurocode designs compared to the CP114 designs regardless of whether the element lengths, design loads, or concrete strengths were changed.

As described in *Section 2.6*, the development of the Eurocodes was an inclusive process and there were multiple individuals involved, from different backgrounds and professions in the construction sector. This led to a holistic view of the design, and an in-depth, comprehensive, set of design codes. This differs from CP114 which is simpler to use, more prescriptive, less performance-based, and created with only structural design considerations.

From the results, it was concluded that the evolution of the design codes led to an unintentional reduction in embodied carbon between the CP114 code and the Eurocodes. However, significantly more can be done in terms of carbon dioxide reductions, both inherently and consciously.

7.2. Further Research and Recommendations

7.2.1. Further Research

The conclusions discussed in *Section 7.1* were determined for simply supported beam and column designs, based on the Eurocodes and Code of Practice CP114. It would be beneficial to investigate similar methodologies and results for different structural elements to determine whether the same conclusions can be applied to these elements. For example, slabs, foundations, piles, etc. should be evaluated. Additionally, an entire building assessment should be conducted to determine the magnitude of the cumulative embodied carbon savings from the evolution of the designs of various elements.

In this research project, only the permanent loads and the live loads were included in the scope of assessment. Wind loads, service loads, and dynamic loads were purposely excluded from the scope in order to not skew the results. Therefore, similar tests or designs must be conducted for external elements that take wind loadings into account and establish whether the results are more or less conservative now or historically. Additionally, dynamic loads, such as earthquakes, etc., and service loads, such as pipes and wiring, should also be considered in a separate study to determine whether this has any significant effect on the current and historic embodied carbon.

Evaluating the embodied carbon from additional historical structural design codes is also crucial in understanding the realistic differences in the embodied carbon over a period of 50 years. This master's research project focussed on Ireland, and it would be valuable to determine if similar results would occur from investigating other geographical regions.

It is suggested that further research is done using beams which are doubly reinforced, as only singly reinforced sections were investigated as part of this study.

Pre-cast elements were not considered or studied as part of this research. It is suggested that this is explored as part of a separate research project which focuses on pre-cast elements as well as a comparison on the embodied carbon reductions or rises from pre-cast members versus in-situ cast members.

The advancements in modern technologies and any efforts to consciously reduce embodied carbon dioxide emissions from the construction sector, such as using SCMs in concrete mix designs, should be considered and the effects of these additions should be quantified in a further study. This should build on from the work started in this research project and consider the effects of optimised dimensions as well as efficient concrete mix designs. This will further establish the reductions in carbon dioxide which are currently possible based on present-day materials.

7.2.2. Recommendations

As seen in this study, the refinement of structural design codes has assisted the construction sector in reducing the embodied carbon dioxide emissions from structures. This is important in order to ensure that reinforced concrete remains the construction material of choice, whilst ensuring that the construction sector does its part in reducing its impacts on climate change as well as meeting the Sustainable Development Goals (SDGs).

However, the extent to which the embodied carbon emissions have been reduced is not enough to meet the levels required in order to slow down and reverse the effects of climate change. The need for efficient structures will become even more necessary in the future. Whilst refining the structural design codes has indeed made some positive impact in reducing embodied carbon emissions, this is incremental and needs to be increased.

As the structural design codes evolve in the near future, dramatic changes to the codes will be required in order to combat climate change. As determined in his research, the volumes of concrete and reinforcement required in structural design are reduced and optimised as our understanding of material properties and our design techniques improve. However, to achieve the carbon dioxide reductions required to combat climate change, there isn't sufficient time to wait for this information and testing. A fundamental change to the structural design philosophies is required. Currently, the widely accepted structural design philosophy is to design and build structures as quickly and as cheaply as possible. This is counter-intuitive to reducing greenhouse gas emissions and material wastage.

The mentality and philosophy behind structural design and construction must shift to efficient structures built with durable materials that have a low embodied carbon and result in little to no wastage.

Currently, there is no requirement in the Eurocodes to make use of Supplementary Cementitious Materials (SCMs). This is a fundamental flaw in the evolution of the Eurocodes. It is suggested that in future structural code iterations that a minimum percentage of cement replacement with SCMs be specified. Rather than requiring a flat rate of cement replacement, which could be too conservative, it would be more beneficial if different cement replacement percentages were required for different structural elements, such as slabs, beams, columns, foundations, etc. This is preferrable that this be a requirement of the structural design codes, rather than the choice of the designer.

Additionally, more emphasis should be placed on the use of prestressed concrete slabs. These can significantly reduce the quantities of materials required in a structure as larger spans are possible, reducing the number of columns required. If precast reinforced concrete elements are used, it is important that the code stipulate higher SCM replacement percentages, as they have time to cure in a controlled environment.

As discussed in Section 2.4, there is a push for reusing and retrofitting existing structures and building. This will prevent the need for large amounts of new virgin materials.

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APPENDIX A: CONCRETE MIX DESIGN PROPORTIONS ENTERED INTO THE EMBODIED CARBON CALCULATOR

Embodied Carbon Model for Concrete

This carbon calculator can be used to calculate the embodied carbon of concrete mixtures. Boundaries are cradle to gate plus the option of transport to site.

1. Concrete Mixture

1a. Mixture Name

	Mix 1	Mix 2	Mix 3
Enter mix Name	Mix 1: Eurocodes	Mix 2: CP114 Mild S	Mix 3: CP114 High Y

1b. Select cement type

SELECT cement type	Cement CEM ...	Cement CEM ...	Cement CEM ...

1c. Enter quantity of above cement type

Cement, *kg per m^3*	310.0	360.0	360.0

1d. Enter any additional cement replacements

GGBS, *kg per m^3*			
Fly ash, *kg per m^3*			
Limestone fines, *kg per m^3*			

For information: Total cementitious content

Cementitious materials, *kg per m^3*	310.0	360.0	360.0
of which:			
Cement	100.0%	100.0%	100.0%
GGBS	0.0%	0.0%	0.0%
Fly Ash	0.0%	0.0%	0.0%

1e. Water to cement ratio

Water	0.55	0.55	0.55

For information: Water content

Water, *kg per m^3*	170.5	198.0	198.0

1e. Admixtures

Average Admixture *kg per m^3 concrete*			
Air entrainers *kg per m^3 concrete*			
Hardening Accelerators *kg per m^3 concrete*			
Plasticisers and Superplasticisers *kg per m^3 concrete*			
Retarders *kg per m^3 concrete*			
Set Accelerators *kg per m^3 concrete*			
Water Resisting Admixtures *kg per m^3 concrete*			

For information: Total admixtures

Total admixtures, *kg per m^3*	0.0	0.0	0.0

1f. Aggregates

Total coarse and fine aggregates, *kg per m^3 concrete*	1,919.5	1,125.5	1,125.5

For information: Total materials

TOTAL material, *kg per m^3 concrete*	2400.0	1683.5	1683.5

2. In-situ or precast

	Mix 1	Mix 2	Mix 3
SELECT concrete type	in-situ	in-situ	in-situ

3. Steel reinforcement

3a. Amount of steel

	Mix 1	Mix 2	Mix 3
Amount of steel *kg / m^3*			

For information: Volume of steel

Volume steel	0.0%	0.0%	0.0%

3b. Type of steel

	Mix 1	Mix 2	Mix 3
Select factor	Custom EPD	Custom EPD	Custom EPD

If you selected custom EPD results in the drop down below, you will need to complete this section. Otherwise, move to the next section. Do not include Mod D. Include Mod A1-3 only

Custom EPD data for steel *GWP - Mod A1-3 - kg CO2e per kg steel*	1.4	1.4	1.4

APPENDIX B: REINFORCEMENT QUANTITIES FOR DESIGN 1

Appendix B.1: Reinforcement Quantity in Simply Supported Beam Design 1

3a Amount of steel

	Mix 1	Mix 2	Mix 3
Amount of steel kg/m^3	99.8	186.9	132.6

For information: Volume of steel

Volume steel	1.3%	2.4%	1.7%

Appendix B.2: Reinforcement Quantity in Column Design 1

3a Amount of steel

	Mix 1	Mix 2	Mix 3
Amount of steel kg/m^3	78.1	68.1	70.8

For information: Volume of steel

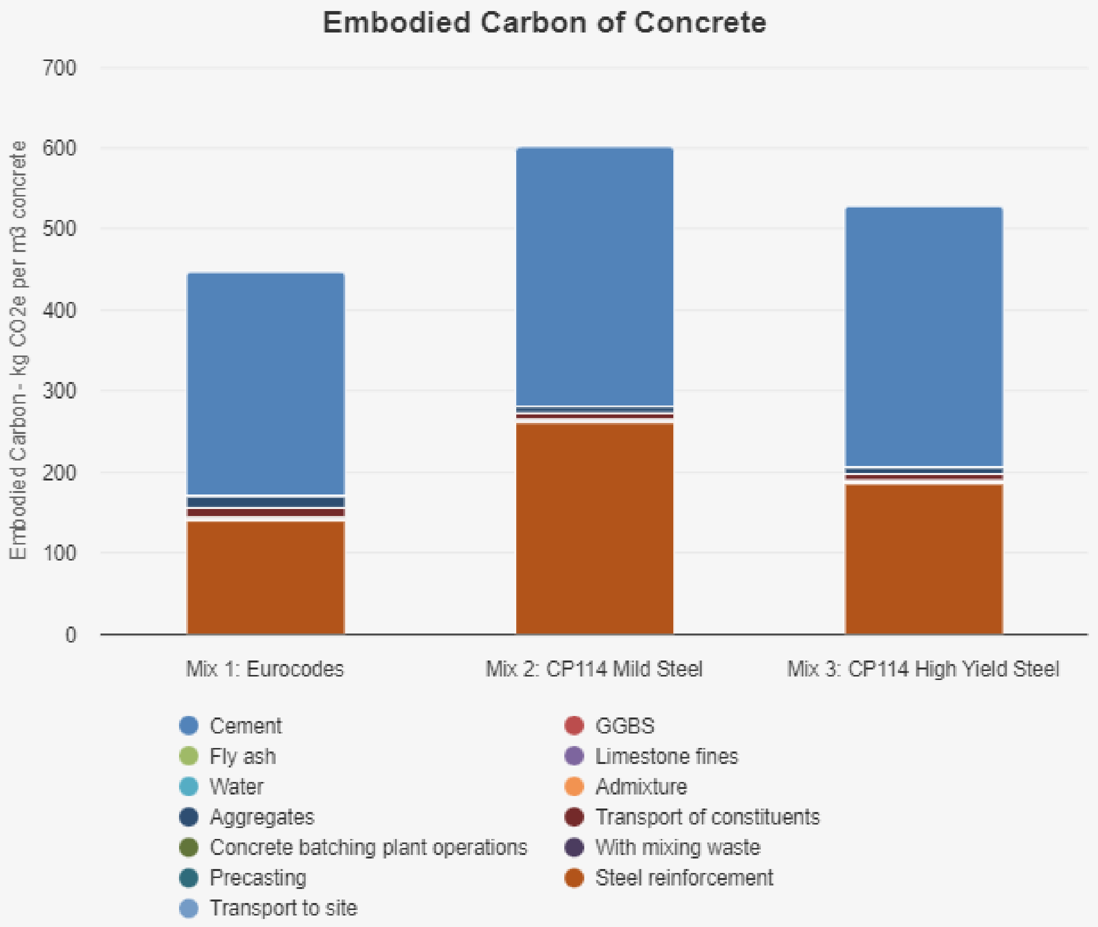
Volume steel	1.0%	0.9%	0.9%

APPENDIX C: EMBODIED CARBON CALCULATOR RESULTS FOR DESIGN 1

Appendix C.1: Simply Supported Beam Design 1 – Results summary from the Embodied Carbon Calculator

5. RESULTS - Summary Charts

A result graph of the three concrete mixtures is shown below:



5. RESULTS - Embodied Carbon

Embodied carbon of concrete per m^3

Material	Mix 1	Mix 2	Mix 3
RESULTS - kg CO2e / m3 concrete	448	602	528

Embodied carbon of concrete per kg concrete

RESULTS - Kg CO2e / Kg Concrete	0.187	0.357	0.314

Results - Contribution %

	Mix 1	Mix 2	Mix 3
Material	Mix 1: Eurocodes	Mix 2: CP114 Mild Steel	Mix 3: CP114 High Yield Steel
Cement	62.3%	53.3%	61.1%
GGBS	0.0%	0.0%	0.0%
Fly ash	0.0%	0.0%	0.0%
Limestone fines	0.0%	0.0%	0.0%
Water	0.0%	0.0%	0.0%
Admixture	0.0%	0.0%	0.0%
Aggregates	3.2%	1.4%	1.6%
Transport of constituents	2.6%	1.4%	1.6%
Concrete batching plant operations	0.4%	0.2%	0.2%
With mixing waste	0.3%	0.3%	0.3%
Precasting	0.0%	0.0%	0.0%
Steel reinforcement	31.2%	43.5%	35.2%
Transport to site	0.0%	0.0%	0.0%

5. RESULTS - Summary Calculations

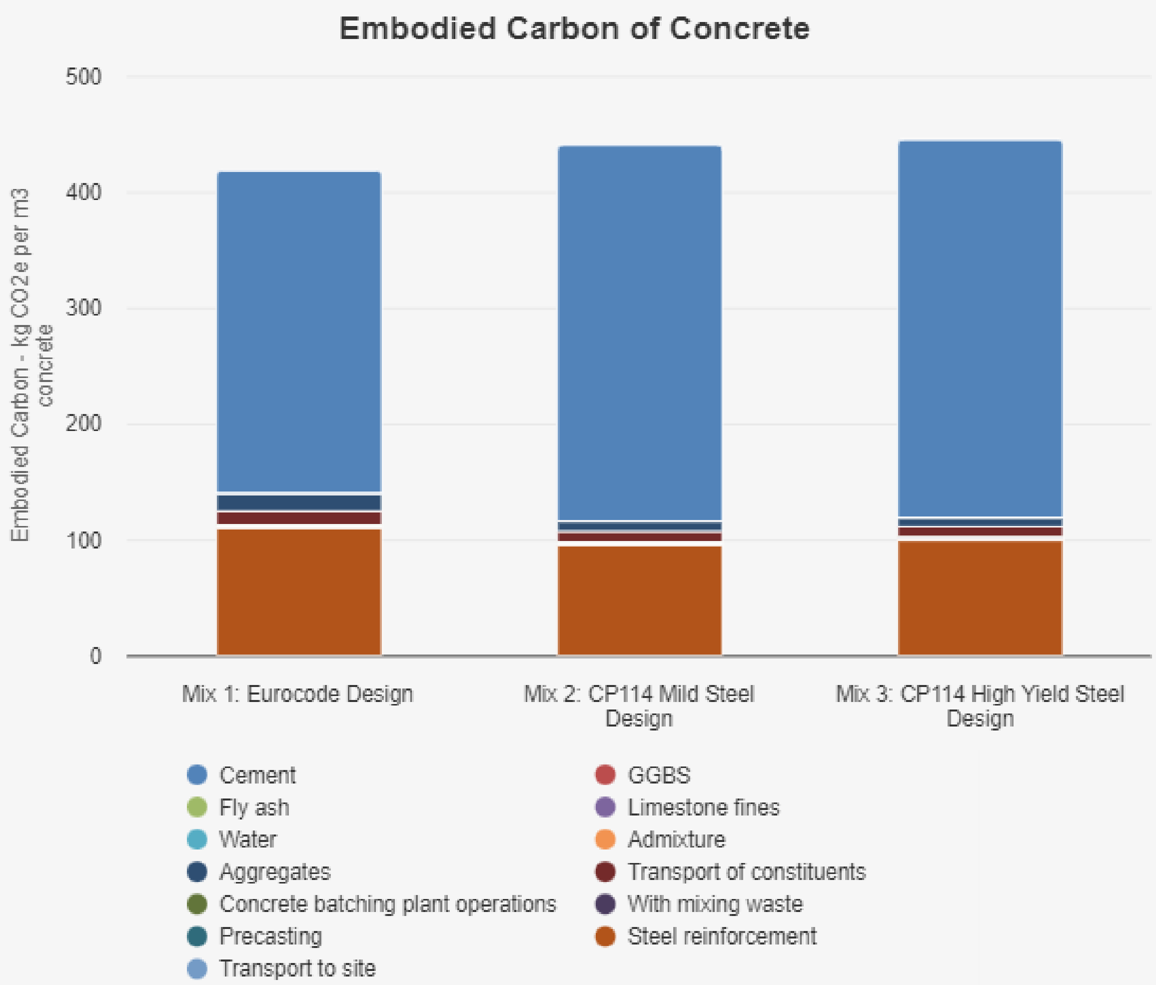
Concrete calculations - Embodied Carbon Contribution - kg CO2e / m^3 concrete

	Mix 1	Mix 2	Mix 3
Material	Mix 1: Eurocodes	Mix 2: CP114 Mild Steel	Mix 3: CP114 High Yield Steel
Cement	279.1	320.5	322.8
GGBS	0.0	0.0	0.0
Fly ash	0.0	0.0	0.0
Limestone fines	0.0	0.0	0.0
Water	0.1	0.1	0.1
Admixture	0.0	0.0	0.0
Aggregates	14.2	8.2	8.3
Transport of constituents	11.8	8.2	8.3
Concrete batching plant operations	1.7	1.2	1.2
With mixing waste	1.53	1.68	1.70
Precasting	0.0	0.0	0.0
Steel reinforcement	139.7	261.7	185.6
Transport to site	0.0	0.0	0.0
RESULTS - kg CO2e / m3 concrete	448	602	528
RESULTS - kg CO2e / kg concrete	0.187	0.357	0.314

Appendix C.2: Column Design 1 - Results summary from the Embodied Carbon Calculator

5. RESULTS - Summary Charts

A result graph of the three concrete mixtures is shown below:



5. RESULTS - Embodied Carbon

Embodied carbon of concrete per m^3

Material	Mix 1	Mix 2	Mix 3
RESULTS - kg CO2e / m3 concrete	419	440	444

Embodied carbon of concrete per kg concrete

RESULTS - Kg CO2e / Kg Concrete	0.174	0.262	0.264

Results - Contribution %

	Mix 1	Mix 2	Mix 3
Material	Mix 1: Eurocode Design	Mix 2: CP114 Mild Steel Design	Mix 3: CP114 High Yield Steel Design
Cement	66.9%	73.9%	73.3%
GGBS	0.0%	0.0%	0.0%
Fly ash	0.0%	0.0%	0.0%
Limestone fines	0.0%	0.0%	0.0%
Water	0.0%	0.0%	0.0%
Admixture	0.0%	0.0%	0.0%
Aggregates	3.4%	1.9%	1.9%
Transport of constituents	2.8%	1.9%	1.9%
Concrete batching plant operations	0.4%	0.3%	0.3%
With mixing waste	0.4%	0.4%	0.4%
Precasting	0.0%	0.0%	0.0%
Steel reinforcement	26.1%	21.6%	22.3%
Transport to site	0.0%	0.0%	0.0%

5. RESULTS - Summary Calculations

Concrete calculations - Embodied Carbon Contribution - kg CO2e / m^3 concrete

	Mix 1	Mix 2	Mix 3
Material	Mix 1: Eurocode Design	Mix 2: CP114 Mild Steel Design	Mix 3: CP114 High Yield Steel Design
Cement	279.9	325.5	325.4
GGBS	0.0	0.0	0.0
Fly ash	0.0	0.0	0.0
Limestone fines	0.0	0.0	0.0
Water	0.1	0.1	0.1
Admixture	0.0	0.0	0.0
Aggregates	14.2	8.3	8.3
Transport of constituents	11.9	8.3	8.3
Concrete batching plant operations	1.7	1.2	1.2
With mixing waste	1.53	1.71	1.71
Precasting	0.0	0.0	0.0
Steel reinforcement	109.3	95.3	99.1
Transport to site	0.0	0.0	0.0
RESULTS - kg CO2e / m3 concrete	419	440	444
RESULTS - kg CO2e / kg concrete	0.174	0.262	0.264

APPENDIX D: RESULTS FOR ADDITIONAL DESIGN CASES

Appendix D.1: Simply Supported Beam Design Results - Eurocodes

Simply Supported Beam Designs (Eurocodes)											
BEAM	Concrete Strength	Reinforcement Strength	Length	Width	Height	Volume	Permanent Load	Variable Load	Total load	Factored design load	Shear Resistance
	fck (MPa)	fyk (MPa)	l (mm)	b (mm)	h (mm)	(m^3)	Gk (kN)	Qk (kN)	(kN)	V,Ed max (kN)	V,rd max (kN)
Beam 1	30/37	500	4000	150	420	0.252	69	3	72	97.7	186.1
Beam 2	30/37	500	4000	250	350	0.350	75	3	78	105.8	252.9
Beam 3	30/37	500	5000	250	450	0.563	112.5	3	115.5	156.4	331.5
Beam 4	30/37	500	6000	250	580	0.870	157.5	3	160.5	217.1	437.8
Beam 5	30/37	500	3000	250	280	0.210	45	3	48.0	65.3	195.6
Beam 6	40/50	500	4000	250	350	0.350	75	3	78	105.8	321.9
Beam 7	28/35	500	4000	250	380	0.380	75	3	78	105.8	261.3
Beam 8	30/37	500	4000	250	440	0.440	135	3	138	186.8	323.3
Beam 9	30/37	500	4000	250	480	0.480	165	3	168	227.3	356.0
Beam 10	30/37	500	4000	250	530	0.530	195	3	198	267.8	396.9

Simply Supported Beam Designs (Eurocodes) - continued						
BEAM	Maximum Design Moment	Resistance Moment	Reinforcement Diameter	Reinforcement Area	Span / Depth	Limiting Span/Dept
	Mmax (kNm)	Mr (kNm)	(mm)	As prov(mm2)	l/d actual	l/d req
Beam 1	97.65	109.71	12	791	10.55	14.64
Beam 2	105.75	115.60	12	1017	12.94	14.74
Beam 3	195.47	229.87	20	1570	12.35	14.41
Beam 4	325.69	368.51	20	1884	11.21	14.62
Beam 5	48.94	52.04	12	565	12.55	16.06
Beam 6	105.75	108.13	12	904	12.94	16.25
Beam 7	105.75	115.41	12	904	11.80	15.30
Beam 8	186.75	223.94	20	1570	10.13	14.40
Beam 9	227.25	246.41	20	1570	9.20	14.38
Beam 10	267.75	278.37	20	1570	8.25	14.61

Appendix D.2: Simply Supported Beam Design Results – CP114

Simply Supported Beam Designs (CP114)										
BEAM	Concrete Strength	Reinforcement Strength	Length	Width	Height	Volume	Dead Load	Live Load	Total Load	Factored design load
	fck (MPa)	fyk (MPa)	l (mm)	b (mm)	h (mm)	(m^3)	DL (kN)	LL (kN)	(kN)	P,LF (kN)
Beam 1: MILD STEEL	30	250	4000	150	630	0.378	67.0	2.7	69.7	125.4
Beam 1: HIGH YIELD STEEL	30	410	4000	150	630	0.378	67.0	2.7	69.7	125.4
Beam 2: MILD STEEL	30	250	4000	250	530	0.530	73.2	2.7	75.9	136.6
Beam 2: HIGH YIELD STEEL	30	410	4000	250	530	0.530	73.2	2.7	75.9	136.6
Beam 3: MILD STEEL	30	250	5000	250	700	0.875	109.5	2.7	112.2	202.0
Beam 3: HIGH YIELD STEEL	30	410	5000	250	700	0.875	109.5	2.7	112.2	202.0
Beam 4: MILD STEEL	30	250	6000	250	900	1.350	153.0	2.7	155.7	280.3
Beam 4: HIGH YIELD STEEL	30	410	6000	250	900	1.350	153.0	2.7	155.7	280.3
Beam 5: MILD STEEL	30	250	3000	250	370	0.278	44.1	2.7	46.8	84.2
Beam 5: HIGH YIELD STEEL	30	410	3000	250	370	0.278	44.1	2.7	46.8	84.2
Beam 6: MILD STEEL	40	250	4000	250	460	0.460	73.2	2.7	75.9	136.6
Beam 6: HIGH YIELD STEEL	40	410	4000	250	460	0.460	73.2	2.7	75.9	136.6
Beam 7: MILD STEEL	28	250	4000	250	520	0.520	73.2	2.7	75.9	136.6
Beam 7: HIGH YIELD STEEL	28	410	4000	250	520	0.520	73.2	2.7	75.9	136.6
Beam 8: MILD STEEL	30	250	4000	250	690	0.690	133.2	2.7	135.9	244.6
Beam 8: HIGH YIELD STEEL	30	410	4000	250	690	0.690	133.2	2.7	135.9	244.6
Beam 9: MILD STEEL	30	250	4000	250	760	0.760	163.2	2.7	165.9	298.6
Beam 9: HIGH YIELD STEEL	30	410	4000	250	760	0.760	163.2	2.7	165.9	298.6
Beam 10: MILD STEEL	30	250	4000	250	830	0.830	193.2	2.7	195.9	352.6
Beam 10: HIGH YIELD STEEL	30	410	4000	250	830	0.830	193.2	2.7	195.9	352.6

Simply Supported Beam Designs (CP114) - continued								
BEAM	Maximum Design Moment	Resistance Moment	Reinforcement Diameter	Reinforcement Area	Concrete Stress	Permissible Concrete Stress	Reinforcement Stress	Permissible Reinforcement Stress
	Mmax (kNm)	Mr (kNm)	(mm)	As prov(mm2)	$\sigma_c(MPa)$	$\rho_{ob}(MPa)$	$\sigma_s(MPa)$	$\rho_{st}(MPa)$
Beam 1: MILD STEEL	125.39	126.15	20	2198	1.327	10	57.05	140
Beam 1: HIGH YIELD STEEL	125.39	126.15	20	1570	1.327	10	79.86	225.5
Beam 2: MILD STEEL	136.62	138.06	24	3164	1.031	10	43.18	140
Beam 2: HIGH YIELD STEEL	136.62	138.06	24	1808	1.031	10	75.56	225.5
Beam 3: MILD STEEL	252.45	256.00	24	4068	1.154	10	49.65	140
Beam 3: HIGH YIELD STEEL	252.45	256.00	24	2712	1.154	10	74.47	225.5
Beam 4: MILD STEEL	420.39	430.56	28	5544	1.246	10	50.55	140
Beam 4: HIGH YIELD STEEL	420.39	430.56	28	3080	1.246	10	90.99	225.5
Beam 5: MILD STEEL	63.18	64.00	20	2198	0.911	10	38.33	140
Beam 5: HIGH YIELD STEEL	63.18	64.00	20	1256	0.911	10	67.07	225.5
Beam 6: MILD STEEL	136.62	141.00	24	3616	1.188	10	37.78	140
Beam 6: HIGH YIELD STEEL	136.62	141.00	24	2260	1.188	10	60.45	225.5
Beam 7: MILD STEEL	136.62	136.68	20	2826	1.051	10	48.34	140
Beam 7: HIGH YIELD STEEL	136.62	136.68	20	1884	1.051	10	72.52	225.5
Beam 8: MILD STEEL	244.62	248.06	24	4068	1.418	10	60.13	140
Beam 8: HIGH YIELD STEEL	244.62	248.06	24	2712	1.418	10	90.20	225.5
Beam 9: MILD STEEL	298.62	306.25	24	4520	1.572	10	66.07	140
Beam 9: HIGH YIELD STEEL	298.62	306.25	24	2712	1.572	10	110.11	225.5
Beam 10: MILD STEEL	352.62	361.00	28	4928	1.699	10	71.55	140
Beam 10: HIGH YIELD STEEL	352.62	361.00	28	3080	1.699	10	114.49	225.5

Appendix D.3: Column Design Results - Eurocodes

Column Designs (Eurocodes)													
COLUMN	Concrete Strength	Reinforcement Strength	Length	Breadth	Height	Volume	Permanent Load	Variable Load	Total load	Factored design load	Axial Load on 3rd floor column	Axial Load on 2nd floor column	Axial Load on 1st floor column
	fck (MPa)	fyk (MPa)	l (mm)	b (mm)	h (mm)	(m^3)	Gk (kN/m)	Qk (kN/m)	(kN/m)	Ed,max (kN/m)	N,Ed,3 (kN)	N,Ed,2 (kN)	N,Ed,1 (kN)
Column 1	30/37	500	3	400	425	0.510	131.25	12.5	143.75	195.9	979.7	1959.4	2939.1
Column 2	30/37	500	3	400	400	0.480	131.25	12.5	143.75	195.9	979.7	1959.4	2939.1
Column 3	30/37	500	2	400	400	0.320	131.25	12.5	143.75	195.9	979.7	1959.4	2939.1
Column 4	30/37	500	4	450	450	0.810	131.25	12.5	143.75	195.9	979.7	1959.4	2939.1
Column 5	40/50	500	3	400	400	0.480	131.25	12.5	143.75	195.9	979.7	1959.4	2939.1
Column 6	28/35	500	3	400	400	0.480	131.25	12.5	143.75	195.9	979.7	1959.4	2939.1
Column 7	30/37	500	3	420	420	0.529	191.25	12.5	203.75	276.9	1384.7	2769.4	4154.1
Column 8	30/37	500	3	440	440	0.581	221.25	12.5	233.75	317.4	1587.2	3174.4	4761.6
Column 9	30/37	500	3	460	460	0.635	251.25	12.5	263.75	357.9	1789.7	3579.4	5369.1

Column Designs (Eurocodes) - continued									
COLUMN	Area Moment of Inertia: Column	Area Moment of Inertia: Beams	Relative Stiffness	Radius of Gyration	Effective Column Length	Slenderness Ratio	Slenderness Limit	Free End Moment: Beam 1	Free End Moment: Beam 2
	I,col (mm^4)	I,beam (mm^4)	k1,k2	i (mm)	$l_0 (m)$	λ	λ_{lim}	$FEM_{beam1} (kNm)$	$FEM_{beam2} (kNm)$
Column 1	2266666667	126562500	7.462	115.470	2.915	25.242	28.180	587.813	261.250
Column 2	2133333333	126562500	7.023	115.470	2.910	25.199	27.338	587.813	261.250
Column 3	2133333333	126562500	10.535	115.470	1.959	16.966	27.338	587.813	261.250
Column 4	3417187500	126562500	8.438	129.904	3.899	30.012	30.756	587.813	261.250
Column 5	2133333333	126562500	7.023	115.470	2.910	25.199	31.568	587.813	261.250
Column 6	2133333333	126562500	7.023	115.470	2.910	25.199	26.411	587.813	261.250
Column 7	2593080000	126562500	8.537	121.244	2.925	24.124	24.145	830.813	369.250
Column 8	3123413333	126562500	10.283	127.017	2.937	23.124	23.626	952.313	423.250
Column 9	3731213333	126562500	12.284	132.791	2.947	22.193	23.261	1073.813	477.250

Column Designs (Eurocodes) - continued							
COLUMN	Floor	Reinforcement Diameter	Reinforcement Chart	Required Reinforcement Area	Provided Reinforcement Area	Minimum Reinforcement Area	Maximum Reinforcement Area
		(mm)		As,req (mm^2)	As,prov (mm^2)	As,min (mm^2)	As,max (mm^2)
Column 1	1st floor	20	0.1	1836.00	1884	225.22	6800
	2nd floor	20	0.1	510.00	628	340	6800
	3rd floor	20	0.1	2244.00	2512	340	6800
Column 2	1st floor	20	0.1	1920.00	2198	225.22	6400
	2nd floor	20	0.1	480.00	628	320	6400
	3rd floor	20	0.1	2304.00	2512	320	6400
Column 3	1st floor	20	0.1	2304.00	2512	225.22	6400
	2nd floor	20	0.1	960.00	1256	320	6400
	3rd floor	20	0.1	2784.00	2826	320	6400
Column 4	1st floor	20	0.1	1215.00	1256	225.22	8100
	2nd floor	20	0.1	0.00	628	405	8100
	3rd floor	20	0.1	607.50	628	405	8100
Column 5	1st floor	20	0.1	1792.00	1884	225.22	6400
	2nd floor	20	0.1	0.00	628	320	6400
	3rd floor	20	0.1	640.00	942	320	6400
Column 6	1st floor	24	0.1	2060.80	2260	225.22	6400
	2nd floor	24	0.1	1344.00	1356	320	6400
	3rd floor	24	0.1	3225.60	3616	320	6400
Column 7	1st floor	28	0.1	3175.20	3696	318.32	7056
	2nd floor	28	0.1	2222.64	2464	352.8	7056
	3rd floor	28	0.1	5821.20	6160	352.8	7056
Column 8	1st floor	32	0.15	4646.40	4824	364.87	7744
	2nd floor	32	0.15	3368.64	4020	387.2	7744
	3rd floor	32	0.15	6969.60	7236	387.2	7744
Column 9	1st floor	32	0.15	4824.48	4824	411.42	8464
	2nd floor	32	0.15	3681.84	4020	423.2	8464
	3rd floor	32	0.15	7998.48	8040	423.2	8464

Appendix D.4: Column Design Results – CP114

Column Designs (CP114)													
COLUMN	Concrete Strength	Reinforcement Strength	Length	Breadth	Height	Volume	Dead Load	Live Load	Total Load	Factored design load	Axial Load on 3rd floor column	Axial Load on 2nd floor column	Axial Load on 1st floor column
	fck (MPa)	fy (MPa)	l (mm)	b (mm)	h (mm)	(m^3)	DL (kN/m)	LL (kN/m)	(kN/m)	P,LF (kN/m)	N,Plf,3 (kN)	N,Plf,2 (kN)	N,Plf,1 (kN)
Column 1: MILD STEEL	30	250	3	675	700	1.418	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 1: HIGH YIELD STEEL	30	410	3	650	700	1.365	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 2: MILD STEEL	30	250	3	690	690	1.428	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 2: HIGH YIELD STEEL	30	410	3	680	680	1.387	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 3: MILD STEEL	30	250	2	690	690	0.952	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 3: HIGH YIELD STEEL	30	410	2	680	680	0.925	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 4: MILD STEEL	30	250	4	690	690	1.904	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 4: HIGH YIELD STEEL	30	410	4	680	680	1.850	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 5: MILD STEEL	40	250	3	690	690	1.428	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 5: HIGH YIELD STEEL	40	410	3	680	680	1.387	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 6: MILD STEEL	28	250	3	690	690	1.428	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 6: HIGH YIELD STEEL	28	410	3	680	680	1.387	126.0	12.5	138.5	249.3	1246.5	2493.0	3739.5
Column 7: MILD STEEL	30	250	3	800	800	1.920	126.0	12.5	198.5	357.3	1786.5	3573.0	5359.5
Column 7: HIGH YIELD STEEL	30	410	3	800	800	1.920	126.0	12.5	198.5	357.3	1786.5	3573.0	5359.5
Column 8: MILD STEEL	30	250	3	860	860	2.219	126.0	12.5	228.5	411.3	2056.5	4113.0	6169.5
Column 8: HIGH YIELD STEEL	30	410	3	860	860	2.219	126.0	12.5	228.5	411.3	2056.5	4113.0	6169.5
Column 9: MILD STEEL	30	250	3	920	920	2.539	126.0	12.5	258.5	465.3	2326.5	4653.0	6979.5
Column 9: HIGH YIELD STEEL	30	410	3	920	920	2.539	126.0	12.5	258.5	465.3	2326.5	4653.0	6979.5

Column Designs (CP114) - continued										
COLUMN	Slenderness Ratio	Slenderness Limit	Reinforcement Diameter	Effective Depth	Factor X	Limiting Load	Permissible Axial Load	Minimum Reinforcement Area	Maximum Reinforcement Area	Provided Reinforcement Area
	λ	λ_{lim}	(mm)	d1 (mm)		Pb (kN)	Po (kN)	As,min (mm^2)	As,max (mm^2)	As,prov (mm^2)
Column 1: MILD STEEL	3.33	15	24	60	0.621	167.719	3814.333	3780	37800	4068
Column 1: HIGH YIELD STEEL	3.46	15	24	60	0.847	167.645	3783.033	3640	36400	4068
Column 2: MILD STEEL	3.26	15	24	60	0.621	164.887	3841.693	3808.8	38088	4068
Column 2: HIGH YIELD STEEL	3.31	15	24	60	0.847	159.921	3839.273	3699.2	36992	4068
Column 3: MILD STEEL	2.17	15	24	60	0.621	164.887	3841.693	3808.8	38088	4068
Column 3: HIGH YIELD STEEL	2.21	15	24	60	0.847	159.921	3839.273	3699.2	36992	4068
Column 4: MILD STEEL	4.35	15	24	60	0.621	164.887	3841.693	3808.8	38088	4068
Column 4: HIGH YIELD STEEL	4.41	15	24	60	0.847	159.921	3839.273	3699.2	36992	4068
Column 5: MILD STEEL	3.26	15	24	60	0.621	164.887	3841.693	3808.8	38088	4068
Column 5: HIGH YIELD STEEL	3.31	15	24	60	0.847	159.921	3839.273	3699.2	36992	4068
Column 6: MILD STEEL	3.26	15	24	60	0.621	164.887	3841.693	3808.8	38088	4068
Column 6: HIGH YIELD STEEL	3.31	15	24	60	0.847	159.921	3839.273	3699.2	36992	4068
Column 7: MILD STEEL	2.81	15	36	90	0.621	270.972	5367.982	5120	51200	9180
Column 7: HIGH YIELD STEEL	2.81	15	36	90	0.847	283.193	5434.486	5120	51200	7140
Column 8: MILD STEEL	2.62	15	36	90	0.621	288.808	6180.940	5916.8	59168	10200
Column 8: HIGH YIELD STEEL	2.62	15	36	90	0.847	292.199	6272.944	5916.8	59168	8160
Column 9: MILD STEEL	2.45	15	36	90	0.621	314.295	6992.620	6771.2	67712	10200
Column 9: HIGH YIELD STEEL	2.45	15	36	90	0.847	326.960	7084.624	6771.2	67712	8160

APPENDIX E: MATERIAL QUANTITIES AND EMBODIED CARBON RESULTS FOR ADDITIONAL DESIGN CASES

Appendix E.1: Simply Supported Beams

Beam	Parameter	Unit	Eurocode Design	CP114: Mild steel design	CP114: High yield steel design
1	Concrete volume (less reinforcement)	m^3	0.259	0.369	0.372
	Reinforcement volume	m^3	0.00316	0.00879	0.00628
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	99.8	186.9	132.6
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	488	602	528
	Total embodied carbon	$kgCO_2e$	121.4	222.3	196.3
2	Concrete volume (less reinforcement)	m^3	0.346	0.517	0.523
	Reinforcement volume	m^3	0.00407	0.00879	0.00628
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	92.3	192.0	108.6
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	438	608	495
	Total embodied carbon	$kgCO_2e$	151.5	314.3	258.9
3	Concrete volume (less reinforcement)	m^3	0.555	0.855	0.861
	Reinforcement volume	m^3	0.00785	0.02034	0.01356
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	111.1	186.8	123.6
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	464	601	516
	Total embodied carbon	$kgCO_2e$	257.5	513.9	444.3
4	Concrete volume (less reinforcement)	m^3	0.859	1.317	1.332
	Reinforcement volume	m^3	0.0113	0.0333	0.0185
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	103.3	198.3	109.0
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	453	617	496
	Total embodied carbon	$kgCO_2e$	389.1	812.6	660.7
5	Concrete volume (less reinforcement)	m^3	0.208	0.271	0.274
	Reinforcement volume	m^3	0.00170	0.00659	0.00377
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	63.9	191.1	108.1
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	399	607	495
	Total embodied carbon	$kgCO_2e$	83.1	164.4	135.5
6	Concrete volume (less reinforcement)	m^3	0.346	0.446	0.451
	Reinforcement volume	m^3	0.00362	0.01446	0.00904
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	81.9	254.8	157.4
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	424	694	562
	Total embodied carbon	$kgCO_2e$	146.9	309.2	253.4
7	Concrete volume (less reinforcement)	m^3	0.376	0.509	0.512
	Reinforcement volume	m^3	0.00362	0.01130	0.00754
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	75.4	174.4	115.4
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	415	585	505
	Total embodied carbon	$kgCO_2e$	156.2	297.6	258.8
8	Concrete volume (less reinforcement)	m^3	0.434	0.674	0.679
	Reinforcement volume	m^3	0.00628	0.01627	0.01085
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	113.7	189.6	125.4
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	467	605	518
	Total embodied carbon	$kgCO_2e$	202.5	407.6	351.8
9	Concrete volume (less reinforcement)	m^3	0.474	0.742	0.749
	Reinforcement volume	m^3	0.00628	0.01808	0.01085
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	104.1	191.3	113.7
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	458	608	502
	Total embodied carbon	$kgCO_2e$	217.0	451.1	376.1
10	Concrete volume (less reinforcement)	m^3	0.524	0.810	0.818
	Reinforcement volume	m^3	0.00628	0.01971	0.01232
	Reinforcement quantity (per m^3 of concrete)	kg/m^3	94.1	191.0	118.3
	Embodied carbon (per m^3 of concrete)	$kgCO_2e/m^3$	440	607	509
	Total embodied carbon	$kgCO_2e$	230.4	491.8	416.2

Appendix E.2: Columns

Column	Parameter	Unit	Eurocode Design	CP114: Mild steel design	CP114: High yield steel design
1	Concrete volume (less reinforcement)	m³	1.530	4.253	4.095
	Reinforcement volume	m³	0.01507	0.03661	0.03661
	Reinforcement quantity (per m³ of concrete)	kg/m³	78.1	68.2	70.8
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	419	440	444
	Total embodied carbon	kgCO2e	634.8	1855.0	1801.9
2	Concrete volume (less reinforcement)	m³	1.440	4.285	4.162
	Reinforcement volume	m³	0.01601	0.03661	0.03661
	Reinforcement quantity (per m³ of concrete)	kg/m³	88.3	67.7	69.7
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	432	440	443
	Total embodied carbon	kgCO2e	622.1	1885.4	1843.6
3	Concrete volume (less reinforcement)	m³	0.960	2.857	2.774
	Reinforcement volume	m³	0.01319	0.02441	0.02441
	Reinforcement quantity (per m³ of concrete)	kg/m³	109.3	67.7	69.7
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	461	440	443
	Total embodied carbon	kgCO2e	442.6	1256.9	1229.1
4	Concrete volume (less reinforcement)	m³	2.430	5.713	5.549
	Reinforcement volume	m³	0.01005	0.04882	0.01627
	Reinforcement quantity (per m³ of concrete)	kg/m³	32.6	67.7	69.7
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	357	440	443
	Total embodied carbon	kgCO2e	867.5	2513.8	2458.2
5	Concrete volume (less reinforcement)	m³	1.440	4.285	4.162
	Reinforcement volume	m³	0.01036	0.03661	0.03661
	Reinforcement quantity (per m³ of concrete)	kg/m³	56.9	67.7	69.7
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	390	440	443
	Total embodied carbon	kgCO2e	561.6	1885.4	1843.6
6	Concrete volume (less reinforcement)	m³	1.440	4.285	4.162
	Reinforcement volume	m³	0.02170	0.03661	0.03661
	Reinforcement quantity (per m³ of concrete)	kg/m³	120.1	67.7	69.7
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	476	440	443
	Total embodied carbon	kgCO2e	685.4	1885.4	1843.6
7	Concrete volume (less reinforcement)	m³	1.588	5.760	5.760
	Reinforcement volume	m³	0.03696	0.08262	0.06426
	Reinforcement quantity (per m³ of concrete)	kg/m³	187.1	114.2	88.6
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	597	503	468
	Total embodied carbon	kgCO2e	947.8	2897.3	2695.7
8	Concrete volume (less reinforcement)	m³	1.742	6.656	6.656
	Reinforcement volume	m³	0.04824	0.09180	0.07344
	Reinforcement quantity (per m³ of concrete)	kg/m³	223.5	109.8	87.6
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	616	497	467
	Total embodied carbon	kgCO2e	1073.3	3308.2	3108.5
9	Concrete volume (less reinforcement)	m³	1.904	7.618	7.618
	Reinforcement volume	m³	0.05065	0.09180	0.07344
	Reinforcement quantity (per m³ of concrete)	kg/m³	214.5	95.8	76.4
	Embodied carbon (per m³ of concrete)	kgCO2e/m³	604	478	452
	Total embodied carbon	kgCO2e	1150.3	3641.2	3443.2

APPENDIX F: ETHICS WAIVER

UNIVERSITY OF THE WITWATERSRAND, JOHANNESBURG

HUMAN RESEARCH ETHICS COMMITTEE (NON-MEDICAL)

Registration number: REC-101114-044

SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING ETHICS COMMITTEE

Ethics clearance / waiver number: CEECC-712668-2023

DATE 23 May 2023

Re: Romana Katrakilis (712668)

To whom it may concern,

Romana Katrakilis (712668) is currently registered as a Masters student at the School of Civil and Environmental Engineering, University of the Witwatersrand, Johannesburg. This letter is to confirm that, at the time of writing, Romana does not need ethical clearance for her study entitled "A Quantitative Investigation of the Carbon Savings in Structural Design based on the Evolution of the Design Codes". This decision has been reached based upon a description of the project supplied by Romana to the School of Civil and Environmental Engineering Ethics Committee, constituted as a subcommittee of the University Human Research Ethics Committee (Non-Medical), which has been evaluated by the subcommittee chair. This decision has then been ratified by the University Human Research Ethics Committee (Non-Medical). If, however, Romana changes the methods of data collection and analysis for this project, this decision may no longer be valid. If such changes take place, this should be communicated to the School of Civil and Environmental Engineering Ethics Committee.

Please feel free to contact me should you require any further information.

Thank you.

Yours sincerely,

Prof John Ndiritu

Shaun Schoeman (Senior Administrative Officer)
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