STRUCTURAL IMPLICATIONS OF REPLACING CONCRETE FLOOR SLABS WITH TIMBER IN COMPOSITE CONSTRUCTION

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DECLARATION STATEMENT:

Kwesi A. Okutu certifies that all the material contained within this document is his own work except where it is clearly referenced to others.
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CHAPTER 1 - Introduction

1.1 Abstract

This project set out to explore the feasibility of using Cross Laminated Timber (XLT) floor slabs in place of precast concrete units in a shallow floor system called Slimflor. As timber is a natural and renewable building resource, this replacement could bring an environmental benefit. It is also much lighter than concrete, meaning it may be possible to reduce the amount of steel used in the frame of a building. To be feasible, any structure built this way must adhere to the requirements of the Building Regulations and the structural design codes (Eurocodes). The most critical aspects to meeting the requirements were anticipated to be ensuring adequate robustness and fire resistance.

In this project, the aforementioned requirements are explored and the ability of the hybrid system to perform was evaluated. The potential environmental benefits were investigated, and a comparative study was performed to gauge the relative performances of the timber and concrete systems. The comparative study looked at how the amount of steel required in the frame, the loads to foundations, the cost, and the environmental impact changed when making the substitution for a range of situations. It found that in buildings with columns closer than 9m apart, and with imposed loads of 4kN/m² or less, using timber meant a reduction in the amount of steel in the frame. The loads to the foundations were reduced significantly in every case studied. The analysis for costs found that using timber would be a more expensive option, mostly due to the relative cost of the slabs themselves (roughly four times the cost of concrete for a given floor area). The study into environmental impact suggested that use of timber was more detrimental to the environment, however this contradicts the literature, and limitations of the calculation model used are noted.

Finally, with the structural performance of the concept verified, some practical considerations were addressed, with suggested methods of enhancing robustness and fire resistance.
1.2 Project Introduction

Sustainability, environmental impact, and efficiency are the focus of increasing concern across wide aspects of contemporary life in the Western world. It is now widely believed that human activity can have a detrimental effect on the natural world, and more stringent laws on industrial activity are being brought in by governments across the world, particularly in the EU. Carbon emissions are one such form of industrial activity that face increasing regulation, and a significant amount of worldwide carbon emissions are as a result of construction processes (5% from cement alone\(^2\)). Businesses will face fines for exceeding limits on carbon emissions, incentivising development of new processes and improvements of existing ones.

In addition to the higher profile of environmental issues, global issues such as financial market instabilities, reducing availability of natural resources and the impact of geopolitical instability on fuel costs mean business and industry are seeking out ways to do more with less, from non-traditional sources. Economics being driver of most decisions, any method of reliably reducing costs will be embraced.

This project explores the potential benefits of a novel adaptation of an existing steel construction technique for composite structures. Standard composite building design makes efficient use of the respective characteristics of steel and concrete as building materials (concrete is good in compression, steel in tension). By joining a steel beam to a concrete slab, composite action means smaller steel sections can be used than otherwise, therefore saving on materials and costs in a variety of ways.

Slimflor is the name of the composite design technique using precast concrete slabs with bespoke Asymmetric Slimflor Beams (ASB). Using this system, the depth of the floor structure can be significantly reduced as the slab rests on the lower flange of the beam rather than on top of the beam. Introduced in 1991\(^3\), this method is now widely used in the UK.

The essence of the new approach is to use the ASBs with prefabricated massive timber slabs of cross-laminated timber (XLT) instead of precast concrete. The hypothesis is that a comparably performing timber slab will be significantly lighter than the equivalent concrete slab, allowing smaller, cheaper section sizes to be used in the building framework. As a result, the environmental impact of a timber floored building will be reduced as the
embodied carbon of the steel frame will reduce with the section sizes, and the significant embodied carbon in the concrete is removed altogether. Concurrently, the cost of the structure will fall, with reduced framework and foundation costs, and quicker rates of construction. Another potential advantage, both economically and environmentally, is that buildings using this technique could be dismantled at the end of the building’s working life, and the slabs (and even frame sections) reused rather than recycled. If such a system can be proven to be feasible, it will be both a financially and environmentally attractive option for steel building design of the future.

1.3 Steel construction in the modern era

1.3.1 Industrial Revolution

The steel we use today is the result of thousands of years of human experimentation, development and refining of metals, but particularly of iron from which it is derived. Our species advanced immeasurably upon the discovery that iron could be extracted from the red ore, and we have never looked back. Developments in metallurgy both lead to and fed from the industrial revolution that began in the late 18th century, and cast iron became suitable for use in construction around 1800[4]. Cast iron was soon joined by wrought iron but by 1890, manufacturing processes had advanced to the point that steel use for construction was ubiquitous. Steel differs from the materials that went before it in the level of control possible in the carbon content and amount and type of impurities present. The percentage of carbon content is what gives steel its appealing qualities, high enough to augment the strength, but not so high as to make it brittle. In the modern world we can add or take away other elements to fine tune the behaviour of the steel as fits the purpose.

Steel sections were and still are hot rolled, with standardisation of sizes beginning in 1901 when BS 1, the first British Standard was published[5].

1.3.2 Composite Construction

Composite action is where two materials of different properties are made to act as one due to being mechanically joined, and the combination of the two materials is better than the sum of the separate parts. In civil engineering, composite construction usually involves having concrete (acting in compression) joined to steel (acting in tension), with both doing
roles they are suited to. One of the earliest forms of composite construction (and remarkably similar in format to shallow floor systems in use today) was the “jack-arch floor”, consisting of cast iron beams supporting masonry vaulting and first used in the 1790s\textsuperscript{5}. Composite floor systems became the topic of interest in the quest to create a fireproof floor. One early example used had iron joists supporting stone floor slabs topped with lime concrete, and once the resistance to fire was confirmed, lime concrete was used in a series of patented fireproof floor designs\textsuperscript{6}. A variety of designs were tried and tested both in the UK\textsuperscript{7} and in the United States of America\textsuperscript{8}, and most involved some form of encasement of the steel beam.

Encasement was taken to the extreme with the advent of filler-joist construction, where a whole iron or steel joist was embedded in concrete to form a flat floor slab\textsuperscript{5} – in many ways this was a precursor to the reinforced concrete design that still exists today.

Composite beam construction of the format now undertaken of steel and concrete has been done since the 1950s\textsuperscript{5}, using reinforcement loops rather than shear studs as connectors between the beam and concrete floor slab.

1.3.3 Slimflor Development

Shallow floor systems are arranged so that the floor slab sits on the extended bottom flange of an asymmetric beam rather than resting on the top flange as is conventional. This means the depth of the flooring system is reduced. The element arrangement enhances the fire performance of the steel beams and connections often without the need for additional protection\textsuperscript{9} because of concrete encasement. Minimising the structural flooring depth has the advantage of allowing services to run in any direction across the building, reduces cladding costs (due to reduced floor-to-floor heights), whilst maximising the working floor height for users. As discussed above, shallow floor systems have been around for a long time, but the idea was revived in the early 1990s. Several concepts were developed and considered, initially built up out of existing sections e.g. the Scandinavian “Thor” beam made from channel sections and two welded plates, and the British “Slimflor” beam consisting of UC section with a single welded plate\textsuperscript{10}. Subsequently, hot-rolled rather than fabricated sections became available. Introduced to the UK in 1997\textsuperscript{11}, the Asymmetric Slimflor Beam (ASB) was originally developed for use with deep decking in \textit{Slimdek} construction\textsuperscript{12}. However, it is now also used with precast units\textsuperscript{13}, which allow even
greater enhancements of fire performance than the deep decking. Further to these advantages, slim floor construction also and allows quick construction due to use of precast elements, whilst still taking advantage of the composite action between the concrete slabs and steel beams\textsuperscript{[13]}.

1.4 – Timber as a construction material

1.4.1 Introduction

Timber is a natural building product, formed from the cutting down and machining of trees. Its origins as a living organism give timber its notable qualities - it is anisotropic, hygroscopic, prone to biological attack and defects, and its mechanical properties vary with species, moisture content and time. However, it is also very strong for its weight, durable, and insulating against heat and sound\textsuperscript{[14]}. When sourced from managed forests, it is also a sustainable, renewable material resource.

1.4.2 Usage variations in time

From the discovery of fire, humans and their predecessors have been using wood and exploiting its properties for millennia. It has been used for many thousands of years for human domiciles, with evidence in China of shelters supported by timber struts dating as far back as 300,000 years, where they were covered in mud possibly to protect the wood from fire\textsuperscript{[15]}.

From the dawn of recorded history, its use has been omnipresent for example the ancient Egyptians used timber for furniture, ornaments and coffins\textsuperscript{[16]}, and the ancient Greeks for beds and boats.

In medieval times, timber started to be used to make bows, and widely for house building in the UK. At the start of the industrial revolution, timber was ubiquitous as a building material, however several disastrous fires in timber framed mill buildings began to change the perception of timber for construction\textsuperscript{[17]}. As time progressed, it was relegated to supporting roofs and floors in smaller domestic settings. However, timber was used extensively elsewhere, for example on the railway network.
1.4.3 Modern Resurgence and Advances

Due to greater awareness of the impact of human actions to the Earth’s ecosystems, timber usage has seen a resurgence due to its sustainability and natural qualities. Technological advances have also yielded more useful wood based products, broadening the potential applications of timber beyond what the original form of the material could achieve.

Engineered Wood Products such as Glue Laminated Timber (GluLam), Laminated veneer lumber (LVL) and cross-laminated timber (XLT) now allow more predictable behaviour and more uses for timber products.

1.4.4 Cross-Laminated Timber

Cross-laminated timber slabs are formed from multiple layers of timber, glued together with each successive layer oriented so the grain direction is perpendicular to the one previous. As a result, the effect of imperfections in the timber such as knots are minimised and balanced out, producing a more uniformly behaving element. Currently, these slabs are produced in Germany, Austria and Switzerland, and can span up to $8\text{m}^{18}$. Slab thicknesses can be produced up to $500\text{mm}$ thick if necessary$^{19}$. These panels can be used to form floors and load bearing walls of a structure, for example Bridport House in Hackney, London. Whilst structures made purely from cross-laminated timber are currently only economically viable to a height of about 8 storeys$^{20}$, recent studies have shown that combining cross laminated timber with a concrete core$^{12}$ or steel support beams$^{13}$, structures of up to 40 storeys are possible. Notable manufacturers of the slab are Massivholz KLH, Binderholz BBS, and MMK, all originating from Austria.

1.4.5 Eurocode Methods

Timber’s immensely variable behaviour means Eurocode 5, the design code for timber structures, incorporates many strength reduction factors to account for the duration of loading (since timber weakens under prolonged loading), the moisture content of the element, the size of the element, and the ability of multiple members to act together. In addition, the anisotropy of timber means a given species of wood has many more strengths than steel or concrete, relating to the direction and type of loading.
As a natural material, both between species and within populations there is great variation in the values of these strengths, meaning design is much more uncertain. Manufactured wood products have slightly more consistent behaviour, but are still subject to safety factors relating to this uncertainty.

Since Cross-Laminated Timber is a relatively new product, it does not have its own set of material and modification factors in the Eurocode. Hence, in the course of this study, when designing to Eurocode requirements, factors relating to Glu-Lam are used as it is the most similar wood-product to cross-laminated timber categorized in ECS.
CHAPTER 2 – Fire Performance

2.1 Introduction

Slimflor using concrete is inherently good at maintaining its structural performance during fires. Conversely the proposed system is based around a well known combustible material. Fire performance is a critical hurdle for the XLT-ASB to deal with, and there are stringent requirements in the design codes to meet. This chapter explores how fire resistance can be addressed.

2.2 Requirements

Design to the Eurocode programme has requirements for the fire performance of a structure. EC0 states:

“In the case of fire, the structural resistance shall be adequate for the required period of time”[21]

EC1 goes on to quote from Construction Products Directive 89/106/EEC on the limitation of fire risks:

“The construction works must be designed and built in such a way, that in the event of an outbreak of fire

– the load bearing resistance of the construction can be assumed for a specified period of time,

– the generation and spread of fire and smoke within the works are limited,

– the spread of fire to neighbouring construction works is limited,

– the occupants can leave the works or can be rescued by other means,

– the safety of rescue teams is taken into consideration”[22].”

The key requirements are to remain adequate for the escape of occupants, the protection of fire-fighting crews, and to prevent further spread of the fire. In order to perform in this way, minimum durations that elements of the structure must retain their strength during a fire are specified by the National Competent Authority, for the UK this is in the Building
Regulations. These minimum durations vary according to building usage, the building size, fire mitigation measures, and the portion of the building under consideration\(^{23}\). It should be noted that the minimum durations stated are not a measure of escape time or how long the structure will remain standing, but merely a way of grouping different structures according to the likely fire loads they would encounter\(^{24}\). Because the size, and the heat intensity of a fire varies considerably due to a great number of reasons, the duration corresponds to the behaviour of an element in a standard test fire – the time it takes for the element to fail during the test fire is then its fire-resistance rating. It can be inferred that if the timber slabs could be made to span further, these savings would be apparent in more cases.\(^{25}\).

In the UK, the limits are set out in Approved Document B of the Building Regulations\(^{26}\). For the type of structures where the proposed structural system could be used, the relevant section is Part B, volume 2. Table A2 outlines the minimum periods of fire resistance according to building usage, whilst Table A1 gives more specific limits according to the section of the building considered. Floors/ceilings have more stringent rating requirements because they serve to delay the spread of fires between floors\(^{26}\).

As the proposed building system is unlikely to be used for industrial sites or storage facilities, the longest fire duration requirement will be 90 minutes.

Construction elements are judged against three criteria regarding fire resistance. These are:

- **Load –Bearing or Stability (R):** the ability to carry the applied load without structural collapse
- **Integrity (E):** Not allowing the fire (hot gases or smoke) to pass through i.e. does not develop cracks or fissures
- **Insulation (I):** Not transferring excessive heat from the side with a fire to the cold side

as given in Approved Document B. The letters in bracket denote the European classification system of the criterion in question. Not all structural elements must satisfy all three requirements – doors for example, are not load bearing so can only be assessed against integrity and insulation. Similarly, beams and columns can only perform in a load-bearing
capacity. Structural floors on the other hand must satisfy all three criteria for the duration of the test to gain that fire-resistance rating (\[^{25}\] Table 6.1).

NOTE – floors/ceilings are tested as exposed to fire from one side (the underside) only. This is because due to convection of fluids the temperature is lower at the floor, and furniture and coverings protect the floor from exposure (\[^{25}\] section 10.7.5).

Whilst the Building Regulations define the minimum period of resistance to fire, it is the Eurocodes that dictate by what means the fire resistance rating is assessed, and how the required minimum resistance can be obtained. For example when designing a concrete beam, Eurocode 2 sets guidelines on the dimensions of the beam and the amount of cover to reinforcement that, if followed, ensure a certain fire rating is achieved.

### 2.3 Timber and Fire Performance

Though the perception is otherwise, timber performs well in fires. Its structural behaviour is easy to predict because it burns at a steady rate, and does not deform because of high temperatures. Charcoal forms on its surface as it burns, and this actually protects the unburned core from damage\[^{27}\]. Though it is well known to aid in the spread of fires and releases heat, it has an ability greater than that of steel to retain its strength under increasing temperatures and advancing time\[^{28}\]. Behaviour in fires varies depending on the species and the moisture content.

#### 2.3.1 The Burning Process

Summarising section 47.4 of Construction Materials: Their Nature and Behaviour, timber does not burn per se, but undergoes thermal degradation when exposed to high temperatures, and in the process emits volatile gases. In the presence of a pilot flame at temperatures above 250°C, or spontaneously above 500°C, the volatile gases ignite and it is the burning of these gases that give the characteristic flame. The thermal decomposition, or pyrolysis, that timber undergoes leads to the blackening of the material, known as charring. The decomposition reaction becomes exothermic as the decomposed timber turns into charcoal and the heat from this induces pyrolysis in the layer beneath, with the char progressing from the surface into the timber. A phenomenon known as transpirational cooling is responsible for the protective behaviour of char – volatile gases emitted from the
pyrolysis zone cool the char layer as they pass through, and are then driven out of the surface in a process that impedes approaching heat waves. This does not stop the progression of heat to the section completely, as char at the surface will fall off, however it does mean a reduced steady state charring rate develops. The char layer has remarkable insulating behaviour to the unburnt layers below. In fire tests of a 160mm deep member, with external temperatures of over 900°C, a 40mm char layer suppressed the temperature of the timber directly below it to 200°C, and to less than 90°C at the middle of the member.[29]

2.3.2 Mechanical Behaviour

During heating, the mechanical properties of timber products are influenced by changes in moisture content, thermal degradation, and charring. Buchanan[25] summarises a number of these considerations.

When the temperature reaches 100°C, the moisture content of the timber changes reduces due to evaporation, and leads to a change in the material behaviour. The combined effects of increased temperature and reduced moisture content are not well understood. Thermal decomposition leads to a loss of strength and charred layers have no strength at all, meaning a reduced effective section size. Heated wood behaves with more plasticity than otherwise – at room temperatures, a beam under bending will fail in a brittle manner on the tension side at a position of weakness such as a knot. Timber is known to perform well in fires, and if there were no plastic behaviour, brittle failure would occur earlier due to the strength reduction and smaller effective section size. Since this is not the observed behaviour, increased plasticity must be allowing more redistribution of stresses within the member.

Structural analysis of timber elements generally takes into account strength reductions and reduced section sizes due to charring.

2.3.3 Manufacturers and Research Findings

According to the Design Brochure for KLH’s slabs, the 5-layer slabs they produce should all be rated to REI 60 or better.[30] For reaction to fire, their slabs are designated according to the European Technical Approval ETA-06/0138 as D-s2, d0 (Combustibility comparable to
wood, medium formation of smoke, produces no flaming debris), but with appropriate coatings could be achieve a rating of B-s1, d0 (Low combustibility, little smoke formation, no flaming debris)\cite{31}.

XLT panels manufactured by KLH have a charring rate of approximately 0.76 mm/min\cite{30}. Experimentation in Switzerland has found that the overall behaviour of cross-laminated timber is very much related to the adhesive used in its manufacture – if the adhesive could prevent charred layers from falling off, the panels behaved like solid wood, however if charred layers could fall off, this allowed much greater charring rates than would be seen in a homogeneous panel\cite{32}. KLH have confirmed the importance of adhesives, as they assert that their panels will meet the integrity criterion provided three layers with two completely intact glue joints remain after exposure to fire\cite{31}. They are bonded using a polyurethane (PU) adhesive, which cures quickly at room temperature. However, the Swiss research suggest that melamine urea formaldehyde (MUF) adhesives are less temperature sensitive, and their use in XLT slab manufacture would improve their fire performance\cite{32}. The negative aspect is that MUF adhesives are less environmentally friendly due to their formaldehyde content.

2.3.4 Verification of Slab Suitability in Fire

In order to assess the behaviour of the floor slabs and calculate they have the required fire resistance, the method from Structural Design for Fire Safety (\cite{25} – Chapter 10) is utilised. This incorporates the Eurocode method of reduced properties for the load-bearing assessment, a check on the time to integrity failure, and the empirical formula for time to failure derived by Janssens\cite{33} for solid wood decks. Janssens’ method may be the most appropriate since it assumes a model of a solid deck consisting of multiple thinner layers – exactly as is the case in reality for cross-laminated timber. The insulation criterion need not be checked if the stability and integrity criteria are met because there will be sufficient floor section left to prevent excessive temperature rises\cite{25}.

*Loading In Fires*

Loading on the structure during a fire is likely to be less than that under normal operation – the permanent load and some of the imposed load. Design codes take account of this fact by reducing the load factors so that for the Eurocode, the design load in fire, $F_{d,f}$, is
\[ F_{d,f} = G_k + 0.5Q_k \quad (2.1) \]

where \( G_k \) is the permanent load and \( Q_k \) is the imposed load \(^{[25]}\) – Table 7.1

**Janssens’ method**

The empirical formula for the time to structural failure, \( t_{sf} \) (minutes), derived by Janssens for design purposes is given by

\[ t_{sf} = 1.25d \left(1 - \sqrt{0.4R_A}\right) - 11.3 \quad (2.2) \]

where \( d \) is the slab thickness (mm), and \( R_A \) is the ratio of applied load (in normal conditions) to design strength. This will assess the load bearing criterion, and is based on transformed section. Here, the applied load used is that of the normal situation.

**Eurocode Method**

The Eurocode Method is a time dependent verification of the element suitability. It assumes the charred section of a structural element has no load bearing capacity, and as such only the uncharred section contributes. The depth of char is calculated at the time duration specified, and the strength of the remaining section can then be found. The depth of char, \( c \), in one dimensional burning is given by

\[ c = \beta t \quad (2.3) \]

where \( \beta \) is the charring rate (mm/min), and \( t \) is the required resistance time (minutes). With this, the reduced section depth, \( d_{r} \), can be found and the reduced section properties (Area and Section Modulus) can be calculated.

The design strength of the remaining section, \( M_f \), is given by

\[ M_f = k_{mod,fi} \cdot \frac{k_{fi}f_pZ_f}{Y_{M,fi}} \quad (2.4) \]

where,

\[ k_{mod,fi} = 1.0 - \frac{1}{200} \frac{p}{A_r} \quad \text{(for bending – ECS part 1-2, 4.2.3(3))} \]

\( p \quad \text{perimeter of cross-section exposed to fire} \)

\( A_r \quad \text{reduced section area} \)
$k_{fi} = 1.15$ for Glulam (Table 2.1 of EC5 part 1-2)

$f_b$ characteristic bending strength

$Z_f$ reduced section modulus

$\gamma_{M,fi}$ partial safety factor for timber in fire ($= 1.0$, EC5 part 1-2, 2.3(1))

This must be greater than the fire loading for the slab to achieve the fire rating.

To check the integrity of the flooring system, EC5 has a further reduction factor to account for the type of joint detail between floor slabs, $\xi$, and it is found from Table E6 of EC5 part 1-2. This factor is applied to the time to burn through the whole slab depth under a notional char rate, $\beta_n$ ($= 0.7\text{mm/min}$ for Glulam – Table 3.1, EC5 part 1-2). Hence the time to integrity failure, $t_{if}$ (minutes), is given by

$$t_{if} = \frac{\xi d}{\beta_n} \quad (2.5)$$

where $d$ is slab depth (mm).

### 2.4 Concrete and Fire Performance

Concrete is an attractive proposition for protecting things from fire since it does not burn nor give off any noxious fumes in high intensity fires. It will eventually lose its strength and start to degrade after a period of exposure, but as explained in "Construction Materials: Their Nature and Behaviour"[28], degradation speed and degree will vary according to many factors such as the element size, temperature gradient, maximum temperature, the constituent parts of the concrete mix and the moisture content.

The degradation process in fires consists of the following:

1. as heating begins, between 20°C and 110°C the concrete effectively dries out as available water is lost through evaporation[16]

2. from around 100°C, water within the structure of the concrete starts to be driven off – if the steam cannot be released quickly enough (for example if the concrete was saturated) pressure will build up in the pores and fissures leading
to cracking and spalling. Concretes of low porosity can suffer explosive spalling because the pressure build up is so great\textsuperscript{128}

3. above 110°C the chemicals and materials of the cement begin decomposing, and as the temperatures increase, the aggregate and cement paste experience high strains at their interface due to differing thermal expansion rates and shrinkage behaviour – above 500°C internal cracking results\textsuperscript{16} and a marked loss of strength is observed\textsuperscript{128}

4. Total loss of strength with the breakdown of hydrates in the cement occurs near 1000°C

As part of a structural element, the concrete will often serve to insulate the more vulnerable reinforcing or prestressing steel. Eurocode 2, part 1-2 specifies a minimum distance from the slab soffit to the reinforcement centreline to prevent the steel reaching the critical temperature of 500°C\textsuperscript{34} within the time of fire resistance specified. Minimum element sizes are also defined to allow for losses of strength – Table 5.8 of EN1992-1-2 contains the values for floor slabs. To prevent explosive spalling from occurring, clause 4.5.1(2) of EN1992-1-2 sets an upper limit on concrete moisture content of k\% by weight, where k varies from country to country (k=3 in the UK)\textsuperscript{35}

**2.5 Slimflor System and Fire Performance**

Unprotected steel sections do not perform well in fires in comparison to those made from concrete or timber. This is due to them being much more conductive to heat, and they usually are thinner. As a material, the stiffness and yield strengths of steel reduce as temperature increases, beginning at about 200°C\textsuperscript{24}, and the nature of the sections leads to quicker increases in temperature. For open sections especially (Universal beams and columns, ASBs etc) heat is able to reach a large proportion of the surface area - for steel beams in fires, the centre of the web is generally the most prone to heating, with the centre of the flanges being the coolest (\textsuperscript{25} – section 8.4). Preventing overheating of the web is key to ensuring the fire resistance of steel members, preventing excessive deformations or catastrophic failure of the beams. If the beam supports a concrete slab on its top flange, this improves the beam’s fire performance as the concrete is able to absorb some of the heat, cooling the beam in the process - the top flange remains cooler than the
bottom flange, allowing the plastic neutral axis to rise and bringing more of the web into tension.\textsuperscript{[24]}

The standard ASB section, when deployed with precast slabs in the Slimflor configuration, is able to resist fires for up to 30 minutes, though the bottom flange can be protected to enhance this figure. Asymmetric Slimflor Beams are also available as ASB(FE) sections. These can achieve fire resistances of up to 60 minutes without protection, due to their thicker webs and so are a valid alternative if the use of protection measures is not possible. This is dependent on sufficient encasement of the beam web being provided\textsuperscript{[13]}.

Encasement is critical to improving the fire performance of the ASBs. In testing, ASBs supporting the deep profiled decking have a shorter duration of fire resistance than those supporting precast concrete units, as the precast units offer more shielding of the beam web\textsuperscript{[13]}. Further protection of the exposed flange may be necessary for higher fire rating durations.

Whilst the performance of the Asymmetric Slimflor Beams may be assured, the performance of the floor as a whole requires further measures. The hollow-core slab units need a combination of special detailing across the ASB and structural in-situ concrete toppings to achieve more than 30 minutes fire resistance. SCI document P342: Design of Asymmetric Slimflor Beams with Precast Concrete Slabs\textsuperscript{[13]} explains the detailing requirements in more depth.

A further consideration is the reduced shear capacity of the hollow core slab units in the fire condition. This is a result of the supporting beams being non-rigid, and liable to deform significantly during a fire, which means less width of the cross section will actually be transmitting the shear loads. SCI-P342 recommends assuming a reduced shear strength of $0.2V_{Rd}$ (where $V_{Rd}$ is the shear capacity of the slab unit from manufacturer’s data) to take this into account.

### 2.6 Connections in Fire

The literature on the Slimflor system does not discuss the behaviour of steel frame connections in fire, though it can be reasonably confidently assumed that connections that are encased in concrete do not need to be considered and that connections not encased
will need protecting in a similar fashion to a steel member. It is considered good practice for connections to be more resistant to fires than the main members themselves\textsuperscript{[25]}. Regarding the proposed timber slab system, the issue to address is the joint of the timber slab and the ASB. Recent experimentation suggests that simple screw fasteners are effective for use securing XLT slabs to a steel frame\textsuperscript{[36]}, thus this method is assumed for the study. What is not known is how well the proposed connection will maintain the integrity of the bay – with time factored deflections and shrinkage of the timber, combined with the deformation of the beam, it is possible that this section could allow smoke and hot gas between floors. Any gaps would lead to an increased char rate at the edge of the timber slabs, causing the gap to increase in size and allowing the propagation of the blaze. This danger is represented in figure 2.1.

A further concern is that the fasteners may cause the XLT slab to fail more quickly. Because metal is such a good conductor of heat, the fasteners may transmit heat deeper into the slab more quickly than the direct charring – if the temperature is great enough, this could hypothetically induce the ignition of the wood in direct contact with the screws, weakening the connection as a result. Whether this phenomenon occurs in reality how integrity is maintained by the joint detail requires verification by experimentation, though the effect of fastener conduction can be minimised by providing fire protection to the exposed ends of the screws.
2.7 Protection Methods

It is worth noting that the behaviour of a structural system in a fire may differ from that of the single elements in fire tests – in the mid 1990s at the Building Research Establishment’s Cardington Laboratory, a building was constructed and set ablaze. In this test, unprotected composite concrete and steel beams withstood temperatures in excess of 1000°C without collapse. This extraordinary behaviour of the overall structure has been attributed to a complex interaction of beam deflection, thermal expansion and the stiffness of the structure as a whole inducing membrane action in the slab\textsuperscript{[24, 25]}. These effects have been incorporated into advanced modelling software such as VULCAN, and allow more situations where fire protection need not be administered\textsuperscript{[24]}. Where fire protection of the steel is required, several types are possible. In the past, the methods most often used on exposed columns and beams were to cover with fire boards or sprayed with protective coatings. Spraying is cheaper, quick to do, and complex shapes can be protected easily, however it is a messy process with an unattractive finish and is liable to be damaged due to its softness\textsuperscript{[25]}. Boards offer a tidier alternative, but they are the most expensive option due to the cost of fitting\textsuperscript{[24]}. Usually made from gypsum, they are good at insulating heat, and pieces can be put between the flanges to create multiple layers of protection to the member web\textsuperscript{[25]}. A more recently available option is Intumescent coating. This is a thin coating that becomes a protective foam only once when heat is applied\textsuperscript{[24]}. These can be specified for the required rating and the application environment, and have become very popular as the finish can be requested during manufacture of the members.

The improved performance of the Slimflor system in fires is due to encasement of the beam in concrete which, as discussed in section 2.5, minimises the need for fire protection. For the viability of the proposed XLT-ASB hybrid, it remains to be ascertained whether the timber slabs could provide similar encasement or protection. When using concrete slabs, the gap between the precast slab and the web of the ASB is filled with in-situ concrete – it is this that ensures the encasement of the web, and the degree of encasement is not possible to replicate when using timber slabs. A possible method of increasing the fire protection to the web is introduced in section 6.3.

The building regulations also involve “reaction to fire”. This is important because rather than just being affected structurally by the fire, the timber slabs will burn themselves, serving to aid the spread of the fire and the development of smoke. The burning of the timber slab at such a proximity will have an effect on the behaviour of the steel ASB,
however the relationship between the timber acting as an insulator from the heat initially to acting as an intensifier as the fire takes hold is unknown and requires further research.

Timber can be treated with fire-retardant chemicals by pressure impregnation. This method is better than surface painting, and aims to slow down the spread of fires across the wood surface\textsuperscript{[25]}. Unfortunately, these chemicals can reduce the strength of the wood, and corrode embedded fasteners. As the timber will still char if temperatures are high enough, none of the current treatments give a quantifiable increase in fire resistance\textsuperscript{[39]}.

2.8 Chapter Conclusion

Emulating the fire performance of Slimflor using precast concrete slabs is a particular challenge for this concept. The inherent encasement concrete offers cannot be easily replicated, and the combustibility of timber raises questions about how beneficial or detrimental the XLT slabs would be. Whilst the integrity of the floor slabs can be maintained in a fairly straightforward manner, the integrity of the connection between slab and ASB is of critical importance to meeting the Eurocode and Building Regulation requirements, and the viability of the scheme altogether.
CHAPTER 3 – Robustness

3.1 Definition

In the common vernacular, to be robust has come to signify being strong or sturdy, however in structural engineering its meaning is much more specific. The “Practical Guide To Structural Robustness and Disproportionate Collapse in Buildings” (page vi) defines robustness as:

“A quality in a structure/structural system that describes its ability to accept a certain amount of damage without that structure failing to any great degree”

In essence, any structural failures should be localised and proportional to the event that caused them, and failures should not propagate through the structure (an event known as a disproportionate collapse). There are similarly worded definitions in the Eurocodes and UK Building Regulations. To understand how this refined definition of robustness came about, it is useful to reflect on the chain of events that led to requirements for robustness entering construction law.

In the aftermath of World War Two, large swathes of housing lay either ruined or severely damaged – around 400,000 homes were destroyed by bombing, and construction and repair of housing effectively ceased during the war[38]. New housing was essential and by the 1951 election it was a key issue. The victorious Conservative government had promised 300,000 new dwellings per year, and over 100,000 council houses were built each year up to 1964. Quantity rather than quality became more important, and councils were under pressure to replace demolished slums, whilst concurrently, the prefabrication system of construction was being developed. Revisions in the subsidy system (Housing Subsidies Act 1956) and the reduced labour costs of using prefabrication encouraged its implementation in high-rise building to such an extent that by 1966, over a quarter of buildings approved by local authorities were blocks of flats of five storeys or more[39]. So when one such block of flats, Ronan Point, suffered a catastrophic collapse as a result of a kitchen oven explosion, there was widespread outrage, fear and pressure on the construction industry to ensure the safety of new buildings. 4 people were killed, however only luck prevented the disaster from having a much greater death toll. It was this event that led to the introduction of robustness requirements.
Ronan Point was made of prefabricated concrete modules stacked on top of one another like building blocks, with only nominal connections to each other laterally. The kitchens were individual modules, each one supporting the kitchens above it. An investigation was swift, and found that the explosion did not destroy the supporting wall or floor, but blew out the wall panel, shearing its connections. With the panel gone, the wall and floor above had nothing to support them and collapsed onto the floor below. The increased weight of the collapsed section overloaded the next floor and this pattern repeated down the building in a progressive collapse.

The investigation recommended that accidental events such as explosions should be considered by designers more extensively, and more specifically recommended stronger tying between structural elements. Clauses were added to the UK design codes to meet these recommendations, and their principles have been carried over into the Eurocode system. Accidental events are covered by Eurocode 1, section 1-7 Actions on structures. General actions. Accidental actions.

3.2. Eurocode and Building Regulation Requirements

The Building Regulations in the UK are the requirements that every building must meet to be deemed safe to use. The Eurocodes form the rules by which a designer must adhere to ensure a building’s structural performance is safe and appropriate. Essentially, the Building Regulations set the requirements and the Eurocodes explain how to achieve them.

In Approved Document A, Section A3, the requirement is set that

“The building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause”[40].

Buildings are categorised according to their purpose and scale, by which the consequence of a disproportionate collapse can be scaled – These are termed “Consequence Classes (CC)”. Larger structures with a greater potential threat or with greater usage by people have a more severe consequence classification. Table 3.1 shows how a building’s consequence class can be ascertained.
<table>
<thead>
<tr>
<th>Class</th>
<th>Building type and occupancy</th>
</tr>
</thead>
</table>
| 1     | Houses no more than 4 storeys tall  
Farming buildings  
Buildings that people do not enter often, provided no part of the building is within a distance of 1.5 times building height from another, more utilised building or area |
| 2A    | Houses of 5 storeys, single occupancy  
Hotels no more than 4 storeys tall  
Offices no more than 4 storeys tall  
Industrial buildings no more than 3 storeys tall  
Educational buildings, single storey  
Retail buildings, no more than 3 storeys tall or 2000m² floor area on any storey  
Any building used by the public no more than 2 storeys tall or 2000m² floor area on any storey |
| 2B    | Residential buildings, flats, apartments, hotels more than 4 storeys tall but no more than 15 storeys  
Retail buildings more than 3 storeys but no more than 15 storeys  
Educational buildings more than 1 storey but no more than 15 storeys tall  
Hospitals no more than 3 storeys tall  
Any building used by the public with more than 2000m² floor area but less than 5000m² on any storey |
| 3     | Any building in categorized above as Class 2A or 2B, but exceeding storey or floor area limits  
Grandstands housing more than 5000 people |

Table 3.1: Building consequence classes (based on Table 11 from Approved Document A of the Building Regulations)
A disproportionate collapse is defined as the collapse of either 15% of the floor area or 70m$^2$ (whichever is smaller) that occurs in two adjacent storeys. The building regulations set this as the maximum permissible level of damage in an accidental event.

The requirement for each class is as follows:

Class 1: No further measures necessary, provided the structure has been built to the relevant design codes and meets the requirements of the other sections of the Building Regulations

Class 2A: Provide effective horizontal ties, or sufficient anchorage of suspended slabs into walls

Class 2B: Provide effective horizontal AND vertical ties. Alternatively, an assessment can be made of the envisioned collapse that would occur with the removal of a each element of the structure individually. If the resulting collapse would be disproportionate (as defined above) then the element must be designed as a key element, together with its connections, to withstand a much greater design load.

Class 3: A comprehensive risk assessment must be made of all potential loads and accidental actions, and design must be carried out accordingly.

It should be noted that Appendix B of Eurocode 1 part 1-7 makes specific reference to unconventional structures (B.9.1(5)) such as those using new materials, as requiring an in depth risk assessment – i.e. they should be in Consequence Class 3. However, it is regarded that the use of timber here (though in a novel format) is suitably conventional, and that the issues investigated and addressed in this document cover the requirements of this clause.

Whilst not mentioned in Eurocode 1 pat 1-7, it is considered good practice to secure floor slabs to their supports, and this connection should be able to hold the self weight of the slab. In the event of an accident, this should reduce the likelihood of floor slabs falling through the steel frame and causing further harm, whilst ensuring the strength of the connection allows the transfer of the self weight to the nearest ties.$^{[41]}$ This recommendation used to be in the British design codes but was not carried over into the Eurocode system.
3.3 Robustness in Comparison Study

Having considered the aspects highlighted in the above sections, the comparative study will include the following measures to take robustness into account in the comparative study.

- Floor slabs will be designed to withstand their self weight as an imposed load as an absolute minimum. This is to meet the no-collapse requirement that prevents progressive failures. In this way, the debris load of the floor above is simulated and designed against to avoid successive floor collapses.

- Horizontal tie beams shall be included in the design so their mass and cost is not overlooked. They shall be designed to withstand tensile forces as stipulated in the Eurocodes (EN1991-1-7, eq. A.1 and A.2)

\[
T_i = 0.8(g_{k} + \psi q_{k})sL \text{ or } 75\text{kN} \quad \text{(whichever is larger)}
\]

\[
T_p = 0.4(g_{k} + \psi q_{k})sL \text{ or } 75\text{kN} \quad \text{(whichever is larger)}
\]

where,

- \(s\) is the distance between ties
- \(L\) is the length of the tie
- \(\psi\) is the combination of action effects factor for accidental loading

3.4 Robustness of the XLT-ASB system

As highlighted in section 3.2, effective anchorage of the floor slab units to the supporting beam is necessary to prevent them falling through the steel frame in the event of an accident. Connecting XLT slabs and steel frames using simple fasteners has been suggested and tested in Canada\(^{[36]}\). For restraining in the direction of span, as is discussed here, the tests demonstrated that simple screw fasteners could withstand up to 17kN each. By appropriate distribution of these fasteners, it is concluded effective anchorage can be achieved. The layout of the slab to frame connection is further explored in Chapter 6.2.
CHAPTER 4 – Disassembly & Reuse

The construction industry faces a confluence of issues relating to man’s interaction with the world and its resources. It is now widely agreed that green house gas emissions (most notably CO$_2$) are causing unpredictable but notable changes in the global climate, and these emissions stem from our ever increasing use of fossil fuels, mostly for energy. However, the reserves are not indefinite, and as the greatest reserves happen to lie in areas that are politically unstable, their availability has become less secure leading to volatile prices.

Against such a backdrop, from a financial, geo-political and ecological perspective, there is great incentive to reduce energy consumption whilst alternative energy sources to fossil fuels are developed. The construction industry uses large amounts of energy in the production of building materials such as steel members, contributing 8-12% of Western Europe’s carbon emissions\(^{[42]}\). Whilst some voluntary reductions in energy use do occur, a financial incentive is a much more effective driver of change than an ecological mindset alone. In this regard, the European Union has created laws and taxes that promote reduced and more ecologically friendly behaviour in the construction sector, for example taxing the use of landfill to encourage more recycling and reuse of building materials\(^{[43]}\) referencing European Commission 2001). Taxes on CO$_2$ emissions are already present on motor vehicles in the UK, with broader carbon taxes likely to appear, which could have an effect on the prices of building materials relative to each other based on the amount of energy required to produce them.

It is also important to mention the significance of public opinion. In the UK at least, the wider public is much more aware of environmental issues, so much so that a company’s environmental credentials are championed and marketed because of the improvement in brand perception and subsequent financial benefit. In construction, this means environmentally friendly solutions can be good for clients with marketing in mind. The environmental benefits of a construction system are now a significant factor in assessing its competitiveness and viability as an alternative to the established building methods and are explored in this chapter.
4.1 Life Cycle Analysis

4.1.1 Definition

A life cycle analysis is a method of analysing a building by the amount of energy used or CO\textsubscript{2} emissions associated with it. There are different methods of performing this life cycle analysis, with varying levels of detail and applicability.

The most basic method is the Cradle to Gate analysis and focuses on the building materials. Here, calculations find the total energy consumption and carbon dioxide emitted to create a unit weight or volume of a given construction material – sometimes referred to as the “embodied energy and embodied CO\textsubscript{2} of the production phase”. This includes the extraction, processing, transportation and all other associated energy and CO\textsubscript{2} up until the point the building material leaves the factory gates.

The next level is the embodied energy of the building. This includes the embodied energy and emissions within the constituent building materials, along with those arising from the construction of the building itself, including the transportation of goods and fitting out.

Once the building enters service the “Operational Phase” begins, and this stage should include any maintenance and repairs. The energy usage and emissions from this stage is usually the largest contributor to the overall analysis.

A full Life Cycle Analysis, or Cradle to Grave analysis includes the embodied energy and CO\textsubscript{2} of the materials, the production phase and the operational phase together, and can be supplemented with the envisaged demolition or deconstruction consumption and emissions.

The most significant elements of a complete life cycle analysis should include manufacture of the building materials and transportation of materials to site (the production phase), maintenance and the day-to-day running of the building in service (operational phase)\textsuperscript{[44]}

4.1.2 Embodied energy vs. Operational energy.

Most of the lifetime energy use is from operational processes, with only 10-15% arising in the production phase of the building\textsuperscript{[44]}.

Operational energy consumption can be reduced through better insulation, natural ventilation schemes etc. However, buildings with low operational energy can have a larger
total energy use due to the increased embodied energy in the machinery necessary to facilitate a reduced operational energy\[44\].

As industry strives to make buildings with a reduced operational energy use, to meet the requirements of the revised Part L of the Building Regulations, the embodied energy of the material choice becomes more significant\[42, 45\]. When combined with the shorter service lives of these buildings, their recycling potential comes into play\[44\].

4.1.3 Energy recuperation through recycling

Recycling of building materials is important and beneficial for managing natural resources that are expensive and energy intensive to extract, however there is an inherent energy cost within the recycling process itself. The efficacy of recycling is also dependent on the quality of the materials involved. Structural steel members, on the other hand, are almost completely (over 90\%\[46\]) recycled as the composition of an old steel section that has been melted down is almost identical to the composition from fresh raw materials.

4.1.4 Reusing components and materials

In reusing products, the extra energy and emissions from the recycling process are avoided, and selling for reuse is more lucrative to the building owner than selling material for scrap. The benefits of reuse and recycling can be used to reduce the embodiment figures for the material in question to more accurately reflect the consumption. Analysis of a one-family house showed a potential embodied energy reduction of 45% through material reuse\[44\]

4.1.5 Keeping disassembly in mind

Acknowledging that operational energy consumption is the most significant, but also the most focussed on presently, Thormark \[45\] concluded that further reducing the life cycle energy use of future buildings consists of the following aspects during design:

- Paying attention to the choice of building materials – their embodied energy and how readily recycled they can be.

- Facilitating future recycling and reuse by allowing for easy disassembly of the structure at the end of its working life.

- Avoiding combinations of materials that will contaminate each other and hinder recycling or reuse efforts.
• Being aware of the contribution maintenance has to operational energy costs – the working life of all parts of the building should be extended wherever possible.

• Novel construction techniques, in terms of material usage and connection design, will encourage greater reductions in embodied energy and greater reuse of materials.

4.2 Timber vs. Concrete – an environmental comparison

Nassen et al\(^{[42]}\), summarising several papers, found that substituting concrete for timber gave significantly reduced CO\(_2\) emissions associated with the structure, and that for this reason, timber as a building material becomes much more competitive when CO\(_2\) emissions are costly, and modelling suggests demand for timber would increase accordingly.

Upton et al. summarising studies\(^{[42]}\):

“For systems with comparable heating and cooling requirements wood-based building systems generally contain lower embodied energy and CO2 emissions than steel, concrete, and brick-based systems.”

The extent of the environmental benefit of timber is of course limited. If there was a shift towards timber as a building material, there would need to be an increase in the amount of forestry that is managed for sustainable growth and high yield. According to Dodoo, Gustavsson and Sathre\(^{[43]}\) summarising Ericksson et al (2007),

“If harvesting levels are increased, age class structure would change towards younger age classes and growth increment would increase. Intensification of forest management on at least part of the forest area, through e.g. fertilization or optimization of thinning operations, would further increase the growth increment, within ecological constraints”

There is only a finite amount of available land for forestry, and a finite capability to manage it in such a manner.

From a recycling perspective, Dodoo, Gustavsson and Sathre\(^{[43]}\) found that the recycling and reuse of timber building products was more beneficial than recycling concrete. After primary use concrete can be recycled by crushing for use as aggregate, but due to the recycling process, compared to fresh aggregate it is difficult to use and would corrode reinforcing steel, thus limiting the level of quality that can be achieved and the range of
applications Timber is currently either reprocessed into new wood based products or burned as a biofuel. Whilst combustion of wood releases CO$_2$, it is better environmentally than fossil fuels because during the tree’s life, it absorbed CO$_2$ during photosynthesis and stored it for many years, and is also a renewable source of energy.

### 4.3 Designing for Disassembly

The potential for disassembly is one of the key advantages of the proposed structural system. Steel frames are sometimes welded together, but they can be bolted instead. This distinction does not have a large impact on disassembly when using a concrete floor slab, because wet concrete is always poured to tie the whole floor structure together. Whilst this is an advantage for the building’s robustness, it makes dismantling the structure significantly more time consuming, difficult and energy intensive. Because the concrete adheres to the steel frame, this limits the amount of steel that can be readily reused, and to recycle the steel or concrete, a large amount of mechanical processing is necessary.

Using a bolted assembly, with a floor slab that is attached but not bonded to the frame, dramatically simplifies the dismantling process, and leaves the structural elements with more potential to be recycled post-use. As confirmed in section 3.4 the XLT slab can be connected to the steel frame by use of simple screw fasteners, and this connection can easily be taken apart.

This capability epitomises the aims and ideals of the Design for Disassembly (DfD) or the equivalent Assembly for Disassembly (ADISA)$^{[47]}$ schools of thought. Essentially, the aim is create buildings that are more easily dismantled for reuse, and with internal components that are more easily replaced when necessary. Simple and easily accessed connections of bolted or screwed assembly are a key recommendation, along with avoiding chemical bonding of elements such as adhesives and bonding agents as these inhibit whole component reuse and make recycling more inefficient.$^{[48]}$

Providing the steel frame is bolted, in theory the entire structural form could be dismantled and then reused or recycled with almost zero wastage. Achieving this is dependent on ensuring the frame and XLT slabs have sufficient durability to last beyond the working life of the building, which is looked at in sections 4.4 and 4.5
EXAMPLES

Temporary and transportable structures most embody the principles outlined in this section, and some of the most high profile examples of temporary structures are Olympic venues. With high capacity buildings and stadia needed, but for a limited amount of time, forward thinking organisers have made use of temporary seating and buildings to avoid so called ‘white elephants’ – expensive projects that have little lasting use beyond the short-term event for which they were originally conceived.

Sydney, Australia 2000 – International Aquatics Centre extension

In order to provide the 17,500 seats required for the hosting of the swimming and diving events, a grandstand of 14,000 extra seats was added to the existing International Aquatics centre. It was designed to be in place for a 12 month period and then dismantled for reuse elsewhere[49]. After the games, the grandstand was relocated 80km to a football stadium. Once disassembled and transported to the town of Wollongong, the grandstand was reconfigured to provide seating for 5000 people at the WIN stadium using more than 80% of the structure from its original setting[50].

London, UK 2012 - Olympic Stadium, Basketball Arena, Aquatics Centre

For the London Olympic Games, sustainability and legacy were core ideals of the project from the outset, setting targets such as reusing, recycling or recovering 95% of demolished materials[51]. To that end, unwanted gas pipes were used in the Olympic stadium’s roof truss compression ring, saving 9 tonnes of embodied carbon[51]. Further, the Olympic stadium was required to seat 80,000 during the games, but be flexible enough to be downscaled to 25,000 seats afterwards. As a solution, a bowl for 25,000 permanent seats was excavated from the ground and 55,000 temporary seats are supported by a demountable steel frame above[52]. The basketball arena is a completely temporary portal frame building, spanning almost 100m and with a capacity of 12,000. The market was studied to ensure there would be demand in the after-market for the components, and with other temporary venues for the project, designed in novel ways to maximise the potential for reuse, sale for hire, and recycling of the constituent parts[53]. The resulting venue has approximately 95% potential for reuse (in weight and value) with the only notable exception being PVC waste pipes due to contamination[53].
The temporary structures built for events such as the Olympics show it is possible to make very efficient and sustainable buildings provided the end of the working life is considered in the initial design. And whilst temporary structures are seen as less durable, as design loads are lower because of the lower statistical likelihood of exceptional loads, the stringent requirements set for the Olympic venues have lead to performances comparable to permanent structures\(^53\). There are notable examples of structures devised as temporary but which have lasted well beyond their intended working life – the London Eye was only meant to be in place for 5 years\(^54,55\), and the Eiffel Tower, now in its 124\(^{th}\) year of service, was only anticipated to stand for 20 years\(^56\).

The case studies show that it is possible to provide demountable solutions. At present they are only used for one off events, but if the thinking behind them was utilised in all structures with some long term thinking, the environmental benefits could be maximised. A set of standardised connections would need to be devised that were mutually compatible and allowed reuse of elements from different sources to connect together simply in the new structural framework.

### 4.4 Reuse of timber

The environmental benefit of using timber slabs rather than concrete depends on the ability of the slabs to be reused at the end of the host structure’s life, which in turn depends on how durable the slabs can be. Discussed in section 5.2, Eriksson et al suggest there would be additional strain put on forest reserves if there were an increased demand for timber as may result from the widespread uptake of the proposed system. However the more modest usage of timber in this system reduces the magnitude of the additional strain put on forest reserves than envisaged Eriksson et al, as they were assuming almost completely timber structures would be used. Maximising the working life (and hence the reusability) of the timber slabs would help further in this issue as there would be less need for fresh specimens.

In KLH’s Environmental Product Declaration According to ISO 14025\(^57\), they state that no maintenance is necessary over the envisaged 50 year working life of the timber slab. In this document they also recommend reuse of the product after this time, or if this is not possible, burning to produce usable energy. The standard assumed working life of most structures is also 50 years\(^{31}\).
The XLT slab can be specified to have surfaces that meet visual quality specifications, and can be used as a working surface. Nevertheless, to improve the recycling potential, treatments are available to protect the timber from deterioration. To guard against fungal and other biological attack, there are a variety of preservative types, including water borne, micro emulsion, and organic solvent preservatives. There are also many different ways of applying the preservatives, depending on the properties of the wood being preserved and the final use e.g. immersion, spraying, or most effectively, pressure impregnation. Further information on this topic can be found in TRADA document WIS 2/-33 “Wood Preservation – chemicals and processes”.

The slabs will also need to be sealed against water infiltration. The edges of the slabs will be particularly susceptible because the end grains are located there and the panel will absorb moisture more readily in this direction due to the microstructure of the material. Absorption of moisture can cause swelling, discolouration, and decay. The swelling will be a particular issue because it will reverse some of the processing that went into making the board meaning that even if the slab were to dry out, the original shape would not be regained\(^\text{[58]}\). Due to the configuration of the flooring system, the ends and sides of the slab units will be inaccessible for maintenance, meaning sealing will have to be done to a very high standard, and measures must be taken to avoid excessive water contact with the slab. This and a variety of edge sealants are discussed in TRADA document WIS 2/3 -20 “Edge Sealants for Wood-Based Boards”.

### 4.5 Reuse of steel

Steel sections can remain in full working order almost indefinitely provided they are protected from corrosion and that this protection is well maintained. Protective coatings are covered by their own European standards, EN ISO 12944 Parts 1 to 8 and EN ISO 14713. With the correct application of an appropriate protective coating for the working environment, the steel section should retain most of its properties.

If the steel is unprotected against corrosion, moisture is an issue for the durability of the steel members. As part of its recommendations for minimising corrosion, the Steel Designers’ Manual advises separating steel and timber either with a coating or plastic sheeting\(^\text{[59]}\), where there is potential for the environment to be wet. It also recommends welds over bolted connections; however the benefits of being able to dismantle the structure at the end of its life outweigh the extra effort that must be taken to prevent corrosion.
With steel members being reused in more situations, the effects of cyclic loading and fatigue will become much more critical in assuring the safe performance of new structures.

Even with flawless maintenance and protection, some frame members may not be able to be directly reused when the building is dismantled especially if they had features bespoke to the original building. In this case, the steel can be recycled, though with lower economic benefit to the client.

### 4.6 Chapter Conclusion

In conclusion to this chapter, there is tremendous potential for the proposed system having a positive environmental impact if replacing concrete as a construction material. The reusability of the XLT-ASB hybrid augments what research suggests is a notable environmental benefit of using timber in preference to concrete. The reusability is dependent on maintaining the durability of the slabs and the steel frame – provided the frame is appropriately protected against corrosion and the slab treated against biological attack and water infiltration, component reuse could be near total, especially considering the shortening working lives of structures.

The temporary structure examples show that the technology and knowledge is there to facilitate component reuse, what is needed now is to take the principles of demountable structure design and create a new normal around them.
CHAPTER 5 – Comparison Study

5.1 Methodology

In order to gauge the effectiveness of using timber, and quantify any possible benefits, a comparative study was performed matching the use of timber slabs against the current Slimflor system with precast concrete units. To get the greatest overview whilst isolating the characteristics in question, the design of a building was pared down to its most basic feature – the sizing of slabs and beams. As much as possible, buildings are set out in regular grids of columns, with each rectangle being known as a bay, and this study involves the design of the elements that constitute a bay. To further simplify the study, the bays being designed are internal bays only. Bays that have edges on the outside of the building are complicated by the presence of facade loads, and the eccentricity of loading on the edge beams – edge beam design is not covered here.

A range of imposed loads are applied for each grid, replicating the loads that may be specified in real applications. Some are matched to building use (where the loads are specified from Eurocode National Annex recommendations).

Requirements from the Eurocode and Building regulations for fire and robustness are incorporated for each building use case.

N.B. Design of columns and vertical ties is omitted in this study.

**Bay Dimensions:** 6m x 9m, 9m x 9m, 12m x 12m

**Imposed Load Cases:**

- Residential: $q_k = 1.5 \text{ kN/m}^2$
- Office: $q_k = 2.5 \text{ kN/m}^2$
- Classroom: $q_k = 3 \text{ kN/m}^2$
- Lecture Hall: $q_k = 4 \text{ kN/m}^2$
- Miscellaneous: $q_k = 6 \text{ kN/m}^2$
- Miscellaneous: $q_k = 8 \text{ kN/m}^2$
- Miscellaneous: $q_k = 10 \text{ kN/m}^2$
The overall study considered the following aspects:

- Finding the optimal beam layout for a given grid and loading
- Slab sizing, including verification for fire, robustness and bearing
- Beam sizing – supporting beams and tie beams
- Steel usage
- Foundation loads
- Composite Behaviour
- Cost analysis
- Environmental Impact

5.2 Beam Layouts

Fair assessment of the timber system required comparing it to the Simflor system both are performing optimally. However, what is optimal? In terms of material usage, cost and emissions, is it better to have a smaller use of steel with large foundations, or use more steel but have smaller foundations?

In this discussion, reduced mass of steel was selected as the measure of optimisation as steel is the most expensive building material per unit mass, and for the quantity surveyors who calculate material costs for proposed schemes, savings in steel will be more noteworthy than a reduction in foundation size.

With the most basic sizing of slabs and beams, layout options of primary and secondary or tie beams were compared, to see which layout yielded the lowest steel usage under the given loading and bay dimensions in the two slab materials.

Using manufacturer’s data (KLH for timber, Bison for hollow-core concrete), slab units were sized according to imposed load and span. Beams were sized according to the required plastic section modulus ($W_{pl}$) and Second Moment of Area ($I_{xx}$) for flexure and deflection satisfaction respectively under the imposed load and self weights. More detailed verification of elements was deemed unnecessary for this section.
Findings

Some grid layouts were not viable because the available slabs would not span that distance under the loading specified, or due to insufficient data from the manufacturers. As the optimal beam layout will be carried into the detailed beam verification, this section therefore mainly serves as the preliminary slab sizing for the detailed design.

In general, having the slabs span as far as possible is the most efficient way of using steel, as the valid solutions using just primary beams and tie beams are always the least consumers of steel. However this does mean much greater foundation loads because the slabs need to be deeper to span the required distance. One of the off-topic conclusions that can be drawn from this portion of the study is that if a proposed structure is constrained in terms of the foundation size, then if feasible, reducing the span of the slab by using secondary beams is the most effective way of reducing the loads to the foundation, as a thinner (and therefore lighter) slab can be used – this holds for both concrete and timber slabs.

A further note is that at this depth of investigation, there is little to be gained from comparing the steel usage of the two slab materials – deflection is the more critical of the sizing criteria for beams in most cases at this stage, and as this is dictated by the imposed load only the chosen beams will be the same for each slab type under identical beam layouts and loading. What can be inferred, however, is the comparative spanning and load carrying capabilities.
### 6m x 9m Grid

#### STEEL USAGE [kg]

<table>
<thead>
<tr>
<th></th>
<th>Residential</th>
<th>Office</th>
<th>Classroom</th>
<th>Lecture Hall</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>6 kN/m²</strong></td>
<td>1967</td>
<td>1478</td>
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#### FOUND. LOAD [kN]

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**Primary Beam**

**Secondary Beam**

**Tie Beam**

---

Fig. 5.1 Layout optimization

6m x 9m grid
### Steel Usage [kg]

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<td>slab</td>
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<td>4118</td>
<td>4118</td>
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<td>6881</td>
<td>7727</td>
<td>9684</td>
<td>10785</td>
<td>13028</td>
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<tr>
<td></td>
<td>4893</td>
<td>5494</td>
<td>7125</td>
<td>7992</td>
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<td>12854</td>
<td>16177</td>
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### Found. Load [kN]

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<tr>
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<td>84</td>
<td>100</td>
<td>108</td>
<td>285</td>
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<td>389</td>
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<td></td>
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<td>267</td>
<td>285</td>
<td>364</td>
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- **Primary Beam**: Reason denotes inability to span
- **Secondary Beam**: ###### best performing
- **Tie Beam**: ###### worst performing

Fig. 5.2 Layout optimization 9m x 9m grid
### STEEL USAGE [kg]

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<td>18053</td>
<td>21692</td>
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### FOUND. LOAD [kN]

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<td>16514</td>
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<td>27737</td>
<td>36371</td>
<td></td>
</tr>
</tbody>
</table>

#### Primary Beam
- 
- 
- 

#### Secondary Beam
- 
- 

#### Tie Beam
- 
- 

---

Fig. 5.3 Layout optimization

12m x 12m grid
5.3 Slab Sizing

Preliminary slab sizing was performed in section 6.2 using load/span charts from slab manufacturers. Other issues also affect the dimensions of the slab, and the preliminary slab choices are hence verified against the following requirements: bearing failure, robustness, and fire-resistance.

5.3.1 Bearing failure

Bearing failure is the localised compression failure of a spanning element or its supports at the area of interface. Designing against this ensures that there is sufficient area over which the element is supported so that the stress there does not exceed the maximum allowable bearing stress.

In timber, the condition to be met for design compressive stress at supports \( (\sigma_{c,90,d}) \) is

\[
\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} \quad (5.1)
\]

\( k_{c,90} \) is a factor depending on the relative positions of the supported and supporting members and also the type of support – this assumes an allowance of increased strain at failure, but can be conservatively taken as 1.0 \(^{[14]}\).

\( f_{c,90,d} \) is the design compressive strength (perpendicular to grain), and is given by

\[
f_{c,90,d} = \frac{k_{mod} k_{sys} f_{c,90,k}}{\gamma_M} \quad (5.2)
\]

where

\( f_{c,90,k} \) characteristc compressive strength (perpendicular to grain)

\( k_{mod} \) modification for load duration and moisture content

\( k_{sys} \) system strength factor

\( \gamma_M \) material partial safety factor (\( = 1.25 \) for Glulam)
For precast concrete slabs, there is a similar requirement on the allowable compressive strength but in Eurocode 2, this is transferred to a recommended nominal bearing length, \( a \), accounting for areas of ineffective bearing and calculated as

\[
a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2}
\]  

(5.3)

here,

\[
a_1 = \frac{F_{Ed}}{(b_1 f_{Rd})}
\]

\(F_{Ed}\) design support reaction

\(b_1\) net bearing width (max 600mm in this scheme)

\(f_{Rd}\) design ultimate bearing strength (=0.4\(f_{cd}\) - dry connection, EC2-1-1 10.9.5.2(2))

\(a_2, a_3\) assumed ineffective distances from the ends of the supported and supporting members respectively (spalling tolerances)

\(\Delta a_2\) to account for construction inaccuracy in the dimension between supports

\(\Delta a_3\) for construction inaccuracy in the length, \( l \), of the supported member (= \( l/2500 \))

Tables of limits on \( a_1 \) and recommended values for \( a_2, a_3, \) and \( \Delta a_2 \) are found in section 10.9.5.2 of EC2 part 1-1.

5.3.2 Robustness

The most onerous requirement on the slab in terms of the robustness requirement is the collapse prevention requirement. Interpreted for design, this means the slab must be able to withstand an imposed load at least as large as the self-weight of the slab so that any collapse of the floor above will not be propagated by failure of the floor below (see section 3.3). For timber, other measures to ensure robustness do not impact significantly on the loading on the steel frame or the subsequent sizing of the members and foundations. However for concrete, the use of an in-situ topping on the precast units helps tie the structure together by producing a continuous slab element. A minimum depth of concrete
must be present over the top flange of the ASB for this to be effective, so the depth of in-situ concrete will impact the weight of the slabs and loads on the beam.

### 5.3.3 Fire Resistance

In timber, fire resistance is the final aspect to be verified, and follows the method outlined in section 2.3.2. Key to the fire-resistance in design is the ξ value in the integrity check, resulting from the type of joint between slabs. It is necessary to prescribe more intricate joints in certain situations to preserve the integrity of the system, and the details of these specifications are found in Table 5.5. Though the fire resistance is based on the situation and loadings, the slab is designed for a 90 minute resistance to maximise the potential reusability, whilst remaining economical. As outlined in section 2.4, the timber can also theoretically protect the steel beam for a period but as the level of performance is unknown and needs research, a fire protection provision is made for the ASB ignoring the presence of the timber slab.

Fire resistance of the concrete slab is assured for the 90 REI rating required providing there is at least 30mm of cover to the pre-stressing strands as discussed in section 2.2 and 2.4. Providing the maximum protection to the ASB, however, requires the encasement of the section with at least 30mm of concrete above the top surface of the upper flange and 50mm of in-situ topping over the concrete slabs themselves. This is to ensure the composite behaviour of the beam and slab, and a 50mm topping is assumed in the sizing tables for the slabs - a maximum of 100mm of topping is practical[61].

The nature of the minimum topping depth requirement means final slab design is derived iteratively in combination with the beam design— the difference between the slab depth and the beam depth must allow for a topping within the prescribed limits, and said beam must be able to support any additional load if the topping is deeper than the assumed 50mm.

N.B: In the absence of the necessary data on the precast slabs, allowing for reduced shear strength of 0.2V_{rd} was not done here as is recommended by SCI document (P351) Precast Concrete Floors in Steel Framed Buildings. Other provisions to meet the 90 REI requirement that do not affect the loading are discussed in section 2.2.

**Findings**

A summary of slab verifications for the timber solution can be found in Table 5.4
### 6 x 9

<table>
<thead>
<tr>
<th>Slab designation</th>
<th>Residential</th>
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<th>Classroom</th>
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<th>6kN</th>
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<td>1.2</td>
<td>1.2</td>
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<td>1.2</td>
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<td>90</td>
<td>90</td>
<td>90</td>
<td>90</td>
<td>90</td>
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<tr>
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<td>Integrity Failure (mins)</td>
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### 9 x 9

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Table 5.4: Slab verification results for XLT slabs
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<td>![Diagram](&lt;2mm 2mm&gt; &gt; 30mm)</td>
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Table 5.5: ξ values according to joint type (based on Table E.6 from BS EN 1995-1-2)
5.4 Beam Sizing

Sizing of the ASBs was performed according to the method set out in SCI document P342 - Design of Asymmetric Slimflor Beams with Precast Concrete Slabs. Initial trial sections were provided by the grid optimisation study.

Composite Behaviour

Composite behaviour is assumed in the concrete solution as is permitted due to the use of a suitable structural topping and detailing. It is only taken into account for serviceability and only in the normal condition. Composite behaviour is also assumed for the timber case, however the accuracy of this assumption is arguable – for simplicity, recommendations for effective breadth of the composite section are assumed the same as for concrete, but this recommendation is based on the shear lag phenomenon which has not been explored with timber or in this configuration.

Tie Beams are sized by calculating the minimum cross-sectional area necessary to prevent yielding of the tie, and to prevent visible sag, a minimum section depth of span/40 is required. Because an internal bay is assumed, design of an internal tie is performed. As discussed in section 3.3, the tie force to be resisted for to ensure robustness, \( T_i \), is given by

\[
T_i = 0.8(g_k + \psi q_k)sL \text{ or } 75\text{kN} \text{ [whichever is larger]} \quad (5.4)
\]

where,

\[
s \quad \text{is the distance between ties}
\]

\[
L \quad \text{is the length of the tie}
\]

\[
\psi \quad \text{is the combination of action effects factor for accidental loading}
\]

Where secondary beams are deployed, they are assumed sufficient to take tying forces without further calculation.

Results of the beam verifications are summarised in Tables 5.6 to 5.8 and form the basis of the sections following.

A fully worked example of the beam and slab verifications is presented in Appendix A.
### Table 5.6: 6m x 9m grid summary

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Table 5.7: 9m x 9m grid summary
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| Steel usage [kg] | 13200 | 15240 | 16512 | 20844 | 27732 | 36372 |
| Foundation Load [kN] | 757 | 1024 | 1174 | 1450 | 2011 | 2602 |
| Composite Enhancement [%] | 3.9 and 1.7 | 2.6 and 5.0 | 2.0 and 5.0 | 1.5 and 2.6 | 1.3 and 1.5 | 0.6 and 1.3 |

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| Steel usage [kg] | 12240 | 12240 | 14040 | 14040 | 16080 | 36372 |
| Foundation Load [kN] | 1780 | 1985 | 2216 | 2432 | 2822 | 3030 |
| Composite Enhancement [%] | 12.9 | 12.9 | 10.9 | 10.9 | 19.3 | NOT PERMITTED |

Table 5.8: 12m x 12m grid summary
### 5.5 Steel Usage

Steel usage is found by summing the mass of primary, secondary and tie beams for a grid once the beam sizing has been verified. By doing this it can be ascertained in which situations the XLT-ASB hybrid structure reduces steel usage.

Figures 5.1 through 5.3 show the mass of steel used by the two construction schemes. Use of the cross laminated timber slabs requires less steel than using concrete, provided the maximum grid dimension is 9m, and the imposed load is 4kN/m$^2$ or less. In all other situations, the precast concrete solution is more economical.

Where the timber solution has the advantage, there is a marked reduction – approximately 15% in the 6m x 9m grid, and 26% for the 9m x 9m grid (on average).

The underlying reason behind this relative behaviour is the differing spanning capabilities of the two types of slab.

Since a concrete hollowcore slab weighs about three times as much as an XLT slab of the same depth, it puts much higher loads on the supporting beams. In preliminary sizing of beams (an output of Section 6.3), it was deflection limits that most influenced beam size, which is independent of slab weight.

However in the detailed design this is not the case. Combined bending and torsion is most
often the critical parameter when sizing the beams for concrete. The substantial reduction in slab mass using timber means ultimate limit states are not critical for beam design, it is working stage deflection as before. The tables summarising beam design show this – the preliminary beam choice has sufficient capacity for all checks in 15 of the 19 cases where timber offers a solution, for concrete it is only 6 out of 20 cases.

This demonstrates that XLT slabs have a latent advantage by introducing reduced stresses into the beam, so a reduction in steel usage as a result of smaller relative section size should be apparent across the board. However, the XLT slabs can span much smaller distances than their concrete counterparts under imposed load – the cases where concrete becomes more economical are when XLT slabs can no longer match the concrete. In the 6m x 9m grid under 6kN/m² of imposed load, the XLT cannot span 9m and so must span across the 6m direction instead, which is a less optimal method as found in section 6.2. On a 9m x 9m grid, this necessitates the use of secondary beams, therefore increasing the amount of steel significantly. On the 12m x 12m grid, secondary beams are required for all loading cases for timber.

This suggests that if the spanning ability of timber slabs can be advanced, the steel usage reduction through use of timber can be extended to more situations.
5.6 Foundation Load Comparison

Loads to columns and foundations are shown in figures 5.4 to 5.6. The values are calculated by multiplying the area of the bay by the design load, comprising of the slab self weight and the imposed load. The steel used in each bay is then added. The use of this value as a column load assumes identical bays in all four quadrants around the column.

What is apparent from the data is that using XLT slab produces a marked reduction in axial loads to columns (and therefore loads to foundations), ranging between 14% and 60%. On average, axial load reduction is 48%. Smaller axial loads lead to a reduction in the size of foundations needed, and because of the lower forces coming from each bay, overturning moments from any asymmetric bay sizes around the column will also be reduced. Reduced loads mean reduced reinforcement is needed, saving costs in material, labour and design.

Foundations represent between 6% and 10% of a structure’s total cost, so this level of reduction would be of significant benefit. Foundations can be complicated by having to negotiate existing groundworks, pipes etc, so reducing their size is also beneficial in terms of ease of design.
Impact on foundation volume

A method of approximating the reduction of the foundation size by using timber has been derived. This assumes a pad foundation for comparison, with no moment in either axis.

To prevent failure in the soil, the limiting requirement is set by

\[
\sigma_{ABP} = \frac{N}{A_{min}} \propto \frac{N}{d_{min}^2} \propto N \sigma_{ABP} \quad (5.5)
\]

where

- \(\sigma_{ABP}\) = Allowable Bearing Pressure of the soil
- \(N\) = Axial load from the column
- \(A_{min}\) = Minimum bearing area
- \(d_{min}\) = Minimum dimension of bearing area

This means that if the axial force in the column is reduced by using XLT relative to concrete, by introducing a scale factor, \(\lambda\), then the reduced bearing area dimension is found as follows.

\[
N_{XLT} = \lambda N_{CONC} \quad (5.6)
\]

\[
d_{min,\text{XLT}}^2 = \lambda d_{min,\text{CONC}}^2 \quad (5.7)
\]

\[
d_{min,\text{XLT}} = \lambda^{1/2} d_{min,\text{CONC}} \quad (5.8)
\]

Transmission of stresses from the edge of the column to the bottom of foundations occurs at a 45° angle. Geometry necessitates a minimum pad depth of \((d_{min} - b_c)/2\), where \(b_c\) is the breadth of the column.

For simplicity of the approximation, minimum pad depth will be taken as \(d_{min}/2\). This is shown in

Fig. 5.7: Nominal pad foundation
Figure 5.7.

The volume of the foundations, Vol\textsubscript{founds}, is then approximated by

\[
\text{Vol}_{\text{founds}} = \frac{d_{\min}^3}{2} \quad (5.9)
\]

Substituting parameters from (5.8),

\[
\text{Vol}_{\text{founds,XLT}} = \frac{d_{\min,XLT}^3}{2} = \left(\frac{\lambda^{3/2}d_{\min,\text{CONC}}}{2}\right)^3 = \lambda^{3/2} \cdot \frac{d_{\min,\text{CONC}}^3}{2} = \lambda^{3/2} \cdot \text{Vol}_{\text{founds,CONC}}
\]

Therefore, when the column loads with a concrete slab are reduced by a factor of 0.5, the foundation volume using timber will be reduced to approximately 0.5\(^{3/2} (= 0.35)\) times the size. This ratio will be used in the cost comparison section.

### 5.7 Composite Behaviour

The calculated enhancements due to assuming composite action of the beam and slab is presented in Tables 5.9 and 5.10. Included is composite enhancement, calculated as the percentage increase in 2\textsuperscript{nd} Moment of Area versus the bare steel beam. Unsurprisingly, due to the stiffer nature of concrete, composite enhancement are greater in the concrete scheme (10% - 45%) when compared to the XLT scheme (0.5% to 12%). Enhancements are generally higher in situations where the use of secondary beams is not required.

The amount of composite enhancement in terms of cm\(^4\) of 2\textsuperscript{nd} Moment of Area is related to four parameters – slab depth, \(d\); beam span, \(L\); width of bottom flange, \(B_b\); and modular ratio, \(n\). All of these values impact on the equivalent area of the composite beam and the composite neutral axis, leading to impact the value of \(I_{\text{comp}}\). Slab depth is the most critical of the values because the additional 2\textsuperscript{nd} Moment of Area is proportional to \(d^3\). This is apparent in the table where, in the 6m x 9m grid for example, the composite enhancement more than doubles with a 50mm increase in slab depth.

Secondary beams are used when the largest slabs available are not able to span the grid under the imposed loads. Only when using timber slabs does composite enhancement apply to use of secondary beams, as the concrete solution does not require them. The reduction in percentage composite enhancement is marked, and has a number of contributing factors. Firstly, the use of secondary beams reduces the distance the slab needs to span, and so a smaller slab section can be used – the reduced depth means a
reduced contribution to composite second moment of area. More important however, is that the primary and secondary beams have to be large sections to span the required distances and meet deflection requirements under the imposed load – their 2nd moments of area are so large to begin with, the enhancement from the timber slab is an order of magnitude lower.

What this suggests is that composite enhancement is most useful for smaller grids and reduced loadings.

There is a balance to find, however. The effective width, $b_{eff}$, is proportional to the spanning length of the beam, so increased span length leads to greater enhancement, though as discussed, timber slab contribution is of most benefit when using the smaller sections. What is clear is that a deeper depth slab will contribute more, raising the possibility that using the deepest slab section possible should be done rather than the smallest. As previously discussed in section 5.5, deflection is the dominant check for timber, and weight of the slab is so low that calculations show using the maximum 300mm slab rather than the minimum XLT slab does not induce failure in any of the cases, but in some cases more than doubling the composite enhancement. Focussing on the cases where timber has already been shown to be more economical, table 5.9 and 5.10 also show the composite enhancements using the 300mm slab. Whilst there will be a price premium to pay on the slab, there is no penalty in steel usage and may in fact reduce the required steel section size. Further, by using a larger slab, this may offer the ASB more fire protection through encasement, though this hypothesis needs to be verified.
<table>
<thead>
<tr>
<th>Loading Case [kN]</th>
<th>Residential</th>
<th>Office</th>
<th>Classroom</th>
<th>Lecture Hall</th>
</tr>
</thead>
<tbody>
<tr>
<td>6m x 9m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Final Beams</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Primary</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Secondary/Tie</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom flange width, Bₙ [mm]</td>
<td>280 ASB 74</td>
<td>280 ASB(FE) 100</td>
<td>280 ASB(FE) 100</td>
<td>280 ASB 105</td>
</tr>
<tr>
<td>Slab depth, d [mm]</td>
<td>165x267x33</td>
<td>165x267x33</td>
<td>165x267x33</td>
<td>165x267x33</td>
</tr>
<tr>
<td>Composite Enhancement [cm⁴]</td>
<td>835</td>
<td>794</td>
<td>1637</td>
<td>2323</td>
</tr>
<tr>
<td>Composite Enhancement [%]</td>
<td>6.9</td>
<td>5.1</td>
<td>10.6</td>
<td>12.1</td>
</tr>
<tr>
<td>Composite Enhancement – 300 mm slab [cm⁴]</td>
<td>2242</td>
<td>2133</td>
<td>2133</td>
<td>2323</td>
</tr>
<tr>
<td>Composite Enhancement – 300 mm slab [%]</td>
<td>18.4</td>
<td>13.8</td>
<td>13.8</td>
<td>12.1</td>
</tr>
</tbody>
</table>

**Table 5.9:** 6m x 9m grid composite enhancement summary

<table>
<thead>
<tr>
<th>Loading Case [kN]</th>
<th>Residential</th>
<th>Office</th>
<th>Classroom</th>
<th>Lecture Hall</th>
</tr>
</thead>
<tbody>
<tr>
<td>9m x 9m</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Final Beams</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Primary</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Secondary/Tie</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bottom flange width, Bₙ [mm]</td>
<td>280 ASB 124</td>
<td>300 ASB 196</td>
<td>300 ASB 196</td>
<td>356 SFB 220</td>
</tr>
<tr>
<td>Slab depth, d [mm]</td>
<td>165x267x33</td>
<td>165x267x33</td>
<td>165x267x33</td>
<td>165x267x33</td>
</tr>
<tr>
<td>Composite Enhancement [cm⁴]</td>
<td>1958</td>
<td>2207</td>
<td>2923</td>
<td>1722</td>
</tr>
<tr>
<td>Composite Enhancement [%]</td>
<td>8.4</td>
<td>4.8</td>
<td>6.4</td>
<td>2.5</td>
</tr>
<tr>
<td>Composite Enhancement – 300 mm slab [cm⁴]</td>
<td>4025</td>
<td>3811</td>
<td>3811</td>
<td>1722</td>
</tr>
<tr>
<td>Composite Enhancement – 300 mm slab [%]</td>
<td>17.6</td>
<td>8.3</td>
<td>8.3</td>
<td>2.5</td>
</tr>
</tbody>
</table>

**Table 5.10:** 9m x 9m grid composite enhancement summary
5.8 Cost Analysis

Methodology

The scope of the cost analysis is limited to the contribution of the structural framework and foundations to the overall cost of the structure. The cases analysed are only those where the XLT solution has been shown to be more economical in terms of steel usage. Using the steel usage data and the floor areas from section 5.4, the cost of the structural framework can be approximated using material costs. The overall cost of constructing the superstructure can then be derived. A relative cost of foundations compared to superstructure yields a cost ratio that is applied to the concrete slab case. Finally, the assumption is made that foundation cost is proportional to foundation volume – the reduction in foundation cost through use of concrete can then be found from by multiplying by $\lambda^{3/2}$, as explained in section 5.6.

Parameters

The costs of materials used in the analysis are presented in table x

<table>
<thead>
<tr>
<th>Component</th>
<th>Indicated Price</th>
<th>Source</th>
<th>Date of Price</th>
<th>Assumed Price</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASBs</td>
<td>£1,080 per tn</td>
<td>TATA\textsuperscript{[62]}</td>
<td>2004</td>
<td>£1,100 per tn</td>
</tr>
<tr>
<td>PC Slabs</td>
<td>£43 per m\textsuperscript{2}</td>
<td>TATA\textsuperscript{[62]}</td>
<td>2004</td>
<td>£45 per m\textsuperscript{2}</td>
</tr>
<tr>
<td>Timber Panels</td>
<td>£120-280 per m\textsuperscript{2}</td>
<td>Building.co.uk\textsuperscript{[63]}</td>
<td>2012</td>
<td>£200 per m\textsuperscript{2}</td>
</tr>
<tr>
<td>Fire Protection (2hr)</td>
<td>£375-500 per tn</td>
<td>Building Magazine\textsuperscript{[66]}</td>
<td>2010</td>
<td>£400 per tn</td>
</tr>
</tbody>
</table>

Table 5.11: Material Costs
Material – Total superstructure scale factor

According to the Steel Designer’s Manual\cite{64},

“Material costs represent only 30-40\% of the total cost of structural steelwork”

Taking 35\% as the material percentage, superstructure cost, $C_{SS}$, is given by

$$C_{SS} = \frac{C_{mat}}{0.35}$$

where $C_{mat}$ is the total cost of materials.

Superstructure – foundation scale factor

From cost models developed by Davis Langdon, it is conservatively assumed that the foundations make up approximately 6.5\% of the total building cost, while the superstructure constitutes 10\%. Hence

$$C_{Founds} = 0.65C_{SS}$$

where $C_{Founds}$ is the cost of foundations

Therefore the transformation factor from material cost, $C_{mat}$, to foundation cost, $C_{Founds}$, is $0.65/0.35 = 1.86$.

$$C_{Founds} = 1.86C_{mat}$$

N.B. This conversion factor applies to the concrete slab solution only. Foundation costs for the XLT solution are derived from the equivalent concrete case and multiplying by the factor $\lambda^{3/2}$, where $\lambda$ is given by

$$\lambda = \frac{N_{XLT}}{N_{CONC}}$$

Fire protection costs are included for the timber case – this is conservative as the protection the XLT slab offers is unknown. As per SCI document P342 (section 6.3), the unprotected steel section can only manage a 30 minute fire resistance. Without confirmation of the XLT slab’s fire encasement abilities, it is advisable to provide fire protection to the beam.
Results

Figures 5.8 and 5.9 show a comparison of the main costs that vary with the choice of slab material.

From the figures, it is clear that the XLT-ASB hybrid structure is the more expensive option within the scope of this study. Using XLT slabs gives cost savings in the steel frame and substantial savings in the foundations, but these savings are offset by the sizeable premium in the cost of the slab. As can be seen in table 5.11 the timber slabs are over 4 times as costly as precast concrete units, becoming the most significant contributor to overall cost in place of the steel frame. If slab costs can be reduced, the system could be much more of a viable alternative to conventional building techniques.

Fig. 5.8: Cost (per bay) breakdown and comparison (6m x 9m)
Validity

With limitations on time and the degree of detail it is possible to go into, this cost model gives only a general sense of the contributory factors to cost. There are other aspects that it does not consider, such as duration of works due to ease of construction, labour costs and transport costs. In addition, the derivation of foundation costs makes liberal approximations that may not accurately reflect the real cost. The costs used for the model are also not as accurate as could be desired due to the absence of readily available and up to date data. For the timber case evidently the cost of the slab is a critical determiner of overall cost, however the suppliers of the XLT slabs were not forthcoming with information on their pricing so the value quoted is for generic engineered laminated timber panels.

With these shortcomings, the model most likely will not accurately predict the cost of a building using the timber or concrete slabs, however it does allow comparisons to be drawn between the systems and demonstrates the critical characteristics.

A more detailed cost analysis and comparison may give different conclusions to the one found here, and would beneficial for clarity on the issue. Cost is a key factor when deciding...
the structural form of a structure meaning unless the cost effectiveness of the XLT slabs can be proven or enhanced, it may not be considered as an option.

5.9 Environmental Impact

The environmental impact of a structure can be measured in many ways, however pursuant to recommendations from Chapter 4, the criteria being assessed are embodied CO₂ and embodied energy. Here, only the steel frame and the slabs themselves are considered, with embodied characteristics being those at the instant of purchase. The material properties are found in table 5.12. The loading cases being considered are the same as for section 5.8 (cost analysis). XLT is considered a glue laminated timber for this section.

<table>
<thead>
<tr>
<th>Material</th>
<th>Embodied Energy [MJ/kg]</th>
<th>Embodied Carbon [kg CO₂/kg]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>1.11</td>
<td>0.159</td>
</tr>
<tr>
<td>Section Steel</td>
<td>21.5</td>
<td>1.42</td>
</tr>
<tr>
<td>Glue Laminated Timber</td>
<td>12</td>
<td>0.87</td>
</tr>
</tbody>
</table>

Fig. 5.12: Material properties for environmental impact[^1]
Results

Fig. 5.10: Environmental Impact summary – Energy Use and CO₂ emissions
Results for the embodied energy and carbon calculation are presented in Figure 5.10. It can be seen that for all cases considered bar one, the concrete solution has a lower embodied energy and lower embodied CO$_2$, despite using a larger quantity of the carbon and energy intensive steel section. As was the case in the cost analysis, it is the slab itself that dominates in carbon and energy cost. The high embodied energy and CO$_2$ of the XLT is likely due to the amount of processing required in their manufacture – tree trunks must be sawn many times, edges and surfaces smoothed before gluing, and heat treatment is applied before delivery. There are also transport costs from the forests to the processing factory to consider which will increase values.

**Validity**

The scope of the study performed here is limited to the Cradle-to-Gate evaluation. Two notable features of the XLT slab are outside of this remit but have a benefit on the environmental impact. Firstly, because when the slab section was part of a tree it took in CO$_2$ in photosynthesis, the absorbed CO$_2$ offsets some of the mass produced in manufacture of the slab, meaning there is a credit that could be applied$^{[57]}$ but is not utilised here. Secondly, if a full Life Cycle Analysis were performed, the ability of the slab to be reused, recycled, or burned to create bio-energy would be taken into account, again lowering the comparative embodied energy and CO$_2$ figures. As discussed in Chapter 5, concrete has less potential for recycling, and hinders the reuse of steel frame members. In a more detailed study, these factors would be included, and hence may give a different outcome. This would explain the discrepancy between findings here and the other research performed into this area and discussed in section 5.2. From this calculation though, the cost premium of the XLT slab over concrete can be somewhat understood, since the cost of the energy used in manufacturing processes are passed on to the product purchaser.
5.10 Chapter Conclusion

This chapter has demonstrated the process of sizing and verifying XLT slabs and ASBs for use in a variety of settings, and how the costs and environmental impacts can be compared against the existing Slimflor system. Uncertainty regarding aspects of the cost model and limited data for the environmental impact of cross laminated timber beyond “cradle-to-grave” has meant definitive conclusions are unable to be made. It is apparent that a reduced amount of steel is necessary to fulfil the structural requirements of a building when using XLT slabs instead of precast concrete for grids of dimension smaller than 9m and imposed loading of 4kN/m² or less. In turn, there is potential for considerable savings to be made in the foundations in those situations. It can be inferred that if the timber slabs could be made to span further, these savings would be apparent in more cases as the need for secondary beams increases steel usage significantly. However the cost of the cross laminated slabs may be prohibitive at this point in time.

Composite action between the cross laminated timber has a beneficial impact on serviceability requirements of the ASB, particularly when using the smaller sections, though this assumes the shear lag effect is of equal effect using timber as it is in concrete – an assumption that is debateable.

Given more reliable and detailed cost, embodied emissions and embodied energy data, the information from the beam sizing and verification could be put to much greater use and extrapolated to approximating the competitiveness of the concept on a real structure. Nevertheless, further information on how the use of timber slabs affects the operational requirements of a building would be useful in gauging the overall economic and environmental impacts.
CHAPTER 6 – Constructability

This chapter looks at the issues relating to the construction of buildings with the ASB-XLT hybrid system, and gives recommendations based on the findings of this report.

6.1 Notching

To facilitate the placement of the slabs during construction, notches may need to be built into the XLT to give clearance to the top flange of the ASB. However, this has implications for the shear capacity of the section – if notches are employed then only the reduced section depth shall be assumed to be acting in shear.

6.2 Connection system

Experimentation by Asiz and Smith\(^{[36]}\) showed that XLT slabs can be secured to steel frame with simple fasteners like screws. Beams can be specified with pre-drilled holes for the screws, and if countersunk screws and holes are used, the bottom of the beam will remain smooth for the application of fire protection or other uses. Self tapping wood screws should be used for maximum interlock between the fastener and the XLT slab. Alternatively, a part coach bolt, part wood screw assembly could be used where greater anchorage is necessary. These are depicted in Figure 6.1. This is a quick and simple method of connection, without the need for specialist machinery, whilst also allowing easy dismantling later. However the suitability of the connection in other respects has not been assured, for example when exposed to damp, or when subjected to dynamic loads. The long term effects on the effectiveness of this connection are also as yet unconfirmed.

![Coach bolt, with nut and Self-tapping wood screws](Fig. 6.1: Suggested anchorage details)
6.3 Integrity in Fires

Because of fire integrity requirements (see section 2.3.4), the slab to slab joint may need to be specified in more detail in certain cases in order to meet Eurocode requirements. This will add to costs, but can be done in the factory for high accuracy. As stipulated by the Eurocodes, gaps for movement between slab units should be a maximum of 2mm wide (see Table 5.5 of this document*). To protect the ASB web during a fire, and to improve integrity between the ASB and the floor slab, a concept using gypsum boards is presented in figure 6.2.

![Gypsum boards and Tape](image)

**Fig. 6.2: suggested integrity measures**

In this concept, an angled sheet of gypsum board is temporarily secured to the upper flange of the ASB before the XLT slab is installed, then released. This gives some extra protection to the beam web and reduces the gap through which smoke or hot gases can pass, without inhibiting the installation process.

6.4 Oversized slabs

As discussed in section 5.7, composite behaviour can be utilised to enhance the serviceability performance of the ASB to good effect when using the smaller ASBs. The largest enhancements were with the deepest slab, and because the increased slab self weight is not critical, it is recommended here to use the deepest slab to maximise composite enhancement. A deeper slab will also give a better overall fire resistance, though there will be a corresponding cost increase.
6.5 Exposure to the elements

KLH state that limited exposure to moisture is permitted during construction, but the slabs must be allowed to dry out thoroughly before use\cite{note1}. Section 4.4 explained how engineered boards are particularly sensitive to swelling due to water infiltration, so every care should be taken to minimise exposure to water. This may mean more stringent storage requirements on site and the use of coverings to protect the slabs. During construction and in service, there is the possibility of exposure to exposure – due to condensation or accidental spillage. It is recommended to separate the timber and steel in some fashion (i.e. they should not bear directly onto one another) as the inherent moisture in timber may cause corrosion of the steel and if additional moisture is present, “nail sickness” may occur in the timber, a chemical decay associated with corroding metal fasteners (\cite{note2} - chapter 46).

A further hazard is the possibility of reversing the seasoning process if the slabs are left exposed for too long. With movements occurring in the timber as moisture content fluctuates\cite{note3}, it may be pertinent to avoid securing the XLT slabs to the framework until the building envelope is completed and the environmental moisture content stabilises to service conditions. Not doing this risks inducing splitting of the boards where the fastener holes have created areas of localised weakness. Doing this will leave the beam unrestrained, so this must be included in the beam verification for the construction cases.
CHAPTER 7 – Conclusions and Future Research

7.1 Conclusion

This study has found that a cross-laminated timber slab can perform the role of a precast concrete slab unit in a comparable manner. For a distinct range of cases (below 4kN/m² imposed load and less than 9m column-to-column distance), the timber system reduces the need for steel. The issues of fire resistance and robustness have been mostly resolved, however the connection between the timber slab and steel beam remains an area of unknowns. Methods were devised to draw comparisons between use of timber and concrete in terms of cost and environmental impact, however their effectiveness and reliability was inhibited by a lack of credible input data.

In conclusion, the feasibility of the system now only rests on ensuring the performance of the slab to beam connection, and the magnitude of the environmental benefits can be determined through more detailed investigation.

7.2 Further Research Areas

This study has covered the feasibility of the XLT-ASB hybrid system, but over the course of the project, other have arisen or been touched upon that it was not possible to fully explore. These areas would benefit from further investigation and are summarised here:

- **Life Cycle Analysis of the system and comparison with conventional Slimflor**
  
  The full story of the associated energy consumption and CO₂ generation was not discerned in this study due to inadequate input data. A full study should be performed, taking into account the possible increased operational costs of using timber\(^{42}\) and the offsets relating to absorbed CO₂ and bio-energy generation.

- **XLT-ASB vs. Standard timber construction**
  
  “For the wood frame construction, the structure is difficult to be reused because this construction technique employs large quantities of adhesive agents, which are difficult to dismantle” – Gao et al.\(^{66}\)
Does the added reusability of the proposed system and the efficiency and predictability of a steel frame make it preferable to timber frame construction?

- **Approximation of Shear lag effects and composite action**
  In composite beams, the equation for effective breadth, $b_{eff}$, given by
  \[
  b_{eff} = \frac{\int_{b_{lr}}^{br} \sigma_x \, dx}{\sigma_{max}}
  \]
  describes dividing the integral of the distribution of horizontal stresses running above and parallel to the composite beam, and dividing by the maximum stress value to get an equivalent rectangular stress block\textsuperscript{[67]}. It is not known whether the stress distribution in concrete-steel composite beams will be the same as for timber-steel composites, meaning assumptions made for composite action by taking concrete’s recommended $b_{eff}$ value may be erroneous. Can composite action be induced using the connection system envisaged?

- **Designing diaphragm action**
  Under horizontal loads such as wind, the floor slab transmits the forces to the columns or bracing elements by acting as a diaphragm. The slab can be assumed to be a horizontal deep beam, a virendeel girder or a diagonal truss. Using these models of the behaviour, the slab and connection have to withstand tension and compression at each side of the deep horizontal beam, shear forces between units, and in combination to transfer loads to the beams and then to the stiff vertical elements. The performance of the hybrid system in this regard is not known.

- **Fire performance of ASBs with timber slabs**
  Timber insulates but is also a source of fire and heat, how does the presence of timber in close proximity help or hinder the performance of the ASB in fire?

- **Enhanced fire performance of ASBs with different adhesives?**
  Research has shown the improved performance of XLT slabs when using formaldehyde based adhesives in manufacture\textsuperscript{[32]}. Do these slabs have a better insulating capacity than those manufactured with polyurethane adhesives?
- **Durability of timber/steel interface and connections in adverse conditions**
  
The connection system tested by Asiz and Smith\(^{36}\) has not been verified against damp conditions, dynamic loading (impact and fatigue), or against creep and time effects that are significant in timber design.


5. M. Bussell, Appraisal of Existing Iron and Steel Structures (P138) (Ascot, UK: The Steel Construction Institute, 1997).


27 'Wis 4-30: Fire Performance of Timber Frame Dwellings', ed. by TRADA (High Wycombe, Buckinghamshire, UK: TRADA Technology Ltd., 2007).
WORKED EXAMPLES

Here are presented the verification process for slabs and beams for the timber slab. For this worked example, the bay size is 9m x 9m and the loading is for an Office.

A.1 GENERAL ARRANGEMENT

![Diagram showing primary beams and tie beams in a 9m x 9m grid]

- **Imposed Load, \( q_k \):** Office (Use category B1) \( 2.5 \text{kN/m}^2 \)
- **Superimposed Permanent Load:** \( 0.5 \text{kN/m}^2 \)

**Properties**

- **Slab designation:** KLH 7ss260
- **Slab depth, \( d \):** 260mm
- **Density, \( \rho_{XLT} \):** 480kg/m\(^3\) = 4.7kN/m\(^3\)
- **Span, \( l \):** 9m
- **Slab unit width, \( b \):** 2.5m
- **Nominal bearing length, \( l_b \):** 80mm

**Slab self weight, \( g_k \):**

\[
g_k = d \cdot \rho_{XLT} + 0.5 = 4.7 \times (260/1000) + 0.5 = 1.72 \text{kN/m}^2
\]

**Design Loads**

\[
\gamma_G = 1.35; \quad \xi = 0.925; \quad \gamma_{Q1} = 1.5; \quad \psi_0 = 0.7
\]

\[
f_{d,1} = \gamma_G \cdot g_k + \gamma_{Q1} \cdot \psi_0 \cdot q_k = (1.35 \times 1.72) + (1.5 \times 0.7 \times 2.5) = 4.97 \text{kN/m}^2
\]

\[
f_{d,2} = \xi \cdot \gamma_G \cdot g_k + \gamma_{Q1} \cdot q_k = (0.925 \times 1.35 \times 1.72) + (1.5 \times 2.5) = 5.90 \text{kN/m}^2
\]
A.2 SLAB BEARING CHECK

Design End shear force, $V_d = \frac{f_{d,\text{max}}b\cdot l}{2} = \frac{5.90 \times 2.5 \times 9}{2} = 66.4\text{kN}$

Design Bearing Stress, $\sigma_{c.90,d} = \frac{V_d}{b\cdot l_b} = \frac{66.4 \times 10^3}{2500 \times 80} = 0.33 \text{ N/mm}^2$

Design bearing Strength, $f_{c.90,d} = \frac{k_{\text{mod},p}k_{\text{sys}}f_{c.90,k}}{\gamma_M}$

$k_{\text{mod},p} = 0.6$  
$k_{\text{sys}} = 1.0$  
$\gamma_M = 1.25$  
$f_{c.90,k} = 2.5 \text{ N/mm}^2$

$f_{c.90,d} = 1.20 \text{ N/mm}^2$

As $\sigma_{c.90,d} (= 0.33) < f_{c.90,d} (= 1.2)$, the slab is verified in bearing.

Allowance for non-rigid supports

$k_{cr} = 0.67$

Design Shear Stress, $\tau_{v,d} = \frac{3}{2} \cdot \frac{V_d}{k_{cr}b\cdot d} = \frac{3}{2} \times \frac{66.4 \times 10^3}{0.67 \times 2500 \times 260} = 0.228 \text{ N/mm}^2$

Design Shear Strength, $f_{v,d} = \frac{k_{\text{mod},p}k_{\text{sys}}f_{v,g,k}}{\gamma_M}$

$f_{v,g,k} = 2.7 \text{ N/mm}^2$

$f_{v,d} = 1.30 \text{ N/mm}^2$

$0.35f_{v,d} (= 0.455 \text{ N/mm}^2) > \tau_{v,d}$ therefore support stiffness does not need consideration.

A.3 FLOOR FIRE PERFORMANCE (ASSUMING 1M STRIP)

XLT flexural strength, $f_{m,k} = 24 \text{ N/mm}^2$

A.3.1 Structural Failure – Janssens’ method (normal loading condition)

Design UDL, $w_d = f_{d,\text{max}} \times 1\text{m} = 5.90 \text{kN/m}$

Design Bending Moment, $M^* = \frac{w_d l^2}{8} = \frac{5.9 \times 9^2}{8} = 59.7 \text{kNm}$

Section Modulus, $Z = \frac{b \cdot d^2}{6} = \frac{1 \times 0.26^2}{6} = 0.0113 \text{ m}^3$
Design Moment Resistance, $M_n = \frac{k_{mod,p} f_{m,k} Z}{\gamma_M} = \frac{0.6 \times 0.0113 \times 24}{1.25} \times 1000 = 130.2 \text{kNm}$

Load ratio, $R_A = \frac{M^*/M_n}{59.7/130.2} = 0.459$

Time to structural failure, $t_{sf} = 1.25d(1 - \sqrt{0.4 R_A}) - 11.3$

$= 1.25 \times 260 \times (1 - \sqrt{0.4 \times 0.459}) - 11.3 = 186 \text{ mins}$

**A.3.2 Eurocode Method – Reduced properties strength verification at 90 minutes**

XLT Char rate, $\beta = 0.76 \text{ mm/min}$

Perimeter exposed to fire, $p = 1 \text{ m}$

Fire load UDL, $w_{d,f} = (g_k + 0.5q_k) \times 1 \text{ m} = 1.72 + (0.5 \times 2.5) = 2.97 \text{ kN/m}$

Design Bending Moment, $M^*_{\text{fire}} = \frac{w_{d,f} \cdot l^2}{8} = \frac{2.97 \times 9^2}{8} = 30.1 \text{ kNm}$

Depth of char (@ $t=90$), $c = \beta \cdot t = 0.76 \times 90 = 68.4 \text{ mm}$

Reduced section depth, $d_f = d - c = 260 - 68.4 = 191.6 \text{ mm}$

Reduced section modulus, $Z_f = \frac{b \cdot d_f^2}{6} = \frac{1 \times 0.1916^2}{6} = 0.0061 \text{ m}^3$

Reduced Section Area, $A_f = b \cdot d_f = 1 \times 0.1916 = 0.192 \text{ m}^2$

Design Strength of Remaining Section, $M_f = k_{mod,fi} \cdot \frac{k_{fi} f_{m,k} Z_f}{\gamma_{M,fi}}$

$k_{mod,fi} = 1.0 - \frac{1}{200 A_r} = 0.974$

$k_{fi} = 1.15$

$\gamma_{M,fi} = 1.0$

$M_f = 164.5 \text{ kNm}$

As $M_f > M^*_{\text{fire}}$, the slab is verified for strength at 90mins fire resistance

**A.3.3 Eurocode Method – Time to integrity failure**

Notional charring rate, $\beta_n = 0.7 \text{ mm/min}$

Time to integrity failure, $t_{if} = \frac{\xi d}{\beta_n}$ for $\xi = 0.2$, $t_{if} = 74 \text{ mins FAILS}$

for $\xi = 0.3$, $t_{if} = 111 \text{ mins OK}$

Therefore, to prevent integrity failure, stipulate lap joints between XLT slabs, with lap dimension greater than 30mm
A.4 BEAM VERIFICATIONS

This verification is based on the methods presented in SCI document P342, with changes to fit the construction scheme and material usage discussed.

Primary Beam: 300 ASB 196

A.4.1 Beam Properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elastic N.A. depth</td>
<td>$y_e$</td>
<td>198 mm</td>
</tr>
<tr>
<td>2nd Moment of Area (Major Axis)</td>
<td>$I_y$</td>
<td>45,871 cm$^4$</td>
</tr>
<tr>
<td>2nd Moment of Area (Minor Axis)</td>
<td>$I_z$</td>
<td>10,463 cm$^4$</td>
</tr>
<tr>
<td>Radius of gyration</td>
<td>$r_y$</td>
<td>6.5 mm</td>
</tr>
<tr>
<td>Torsional Constant</td>
<td>$I_T$</td>
<td>1177 cm$^4$</td>
</tr>
<tr>
<td>Elastic Modulus (top)</td>
<td>$W_{el,y}$</td>
<td>2321 cm$^3$</td>
</tr>
<tr>
<td>Plastic Modulus</td>
<td>$W_{pl,y}$</td>
<td>3055 cm$^3$</td>
</tr>
<tr>
<td>Cross Section Area</td>
<td>$A$</td>
<td>249 cm$^2$</td>
</tr>
<tr>
<td>Warping Constant</td>
<td>$I_W$</td>
<td>1,500,000 cm$^6$</td>
</tr>
<tr>
<td>Buckling parameter</td>
<td>$u$</td>
<td>0.845</td>
</tr>
<tr>
<td>Torsional Index</td>
<td>$x$</td>
<td>7.86</td>
</tr>
<tr>
<td>Flange width (top)</td>
<td>$B_t$</td>
<td>183 mm</td>
</tr>
<tr>
<td>Flange Width (bottom)</td>
<td>$B_b$</td>
<td>293 mm</td>
</tr>
<tr>
<td>Depth of Section</td>
<td>$D$</td>
<td>342 mm</td>
</tr>
<tr>
<td>Web thickness</td>
<td>$t_w$</td>
<td>20 mm</td>
</tr>
<tr>
<td>Flange thickness</td>
<td>$t_f$</td>
<td>40 mm</td>
</tr>
<tr>
<td>Root Radius</td>
<td>$r$</td>
<td>27 mm</td>
</tr>
</tbody>
</table>

Section is Class 1

Beam Spanning distance         9m
Beam Spacing (Slab span)        9m
Beam weight = 196kg/m = 1.92kN/m
A.4.2 Construction Stage

Imposed Load = Construction Load = 0.5 kN/m²

Case 1: (XLT slab + Construction Loading), on one side of ASB only.

Assume bearing length of 40mm for design

![Construction Loading Diagram]

Imposed UDL, q_k = 0.5 x 4.5m = 2.25 kN/m

Permanent Load (slab) = Slab unit weight = 4.7 x (260/1000) = 1.22 kN/m²

Permanent Load (beam) = 1.92 kN/m

Permanent UDL, g_k = slab + beam = (1.22 x 4.5m) + 1.92 = 7.41 kN/m

\[ f_d.1 = \gamma_G g_k + \gamma_Q q_k = (1.35 \times 7.41) + (1.5 \times 0.7 \times 2.25) = 12.4 \text{ kN/m} \]

\[ f_d.2 = \xi \gamma_G g_k + \gamma_Q q_k = (0.925 \times 1.35 \times 7.41) + (1.5 \times 2.25) = 12.6 \text{ kN/m} \]

\[ f_d = \max(f_d.1; f_d.2) = 12.6 \text{ kN/m} \]

Lateral Torsional Buckling (LTB) for combined bending and tension

Verify against interaction formula:

\[ \frac{M_y m_{LT}}{M_{b,Rd}} + \frac{\sigma_{bzt} + \sigma_w}{f_y} \left( 1 + \frac{0.5 \cdot M_y m_{LT}}{M_{b,Rd}} \right) \leq 1 \]

Major axis moment, \( M_y \) = \[ \frac{f_d l^2}{8} = \frac{12.6 \times 9^2}{8} = 128 \text{ kNm} \]

Torsion Load = 4.5m x 9m x [(0.925 x 1.35 x 1.22) + (1.5 x 0.5)] = 92.1 kN

Torsion Lever arm = \[ \frac{B_b - 40}{2} = \frac{293 - 40}{2} = 126.5 \text{ mm} \]

Applied torsion, \( T_q \) = 92.1 x (126.5/10³) = 11.7 kNm
Finding buckling resistance of the section, $M_{b,Rd}$:

First, elastic critical moment ($M_{cr}$) is given by

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{L_{cr}^2} \left[ \frac{I_w}{I_z} + \frac{L_{cr} G I_T}{\pi^2 E I_z} \right]^{0.5}$$

- $C_1 = 1.132$ (simply supported beam under UDL)
- $L_{cr} = 9m$ (beam assumed unrestrained during construction)

$$M_{cr} = 1.132 \times \frac{\pi^2 \times (210 \times 10^9) \times \left(\frac{10,463}{10^8}\right)}{9^2} \times \left[ \frac{\left(\frac{1,500,000}{10^{12}}\right)}{\pi^2 \times (210 \times 10^9) \times \left(\frac{10,463}{10^8}\right)} \right]^{0.5}$$

$$M_{cr} = 1,844,561 \text{ Nm} = 1,845 \text{ kNm}$$

Buckling parameter, $\lambda_{LT}$ is then given by

$$\lambda_{LT} = \sqrt{\frac{W_y f_y}{M_{cr}}}$$

- $W_y = W_{pl,y}$ as section is Class 1
- $f_y = 355 \text{ MPa}$

$$\lambda_{LT} = \sqrt{\frac{3055 \times 355}{1845 \times 1000}} = 0.767$$

$D/B_t = 342/183 = 1.87 < 2$, therefore Imperfection Factor, $\alpha_{LT} = 0.21$

Now, reduction factor for LTB, $\chi_{LT}$ is given by

$$\chi_{LT} = \frac{1}{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}} \leq 1.0$$

Here,

$$\phi_{LT} = 0.5 \left[ 1 + \alpha_{LT} (\lambda_{LT} - 0.2) + \lambda_{LT}^2 \right] = 0.853$$

Therefore,

$$\chi_{LT} = \frac{1}{0.853 + [0.853^2 - 0.767^2]^{0.5}} = 0.814$$

Finally, design buckling resistance of unrestrained beam, $M_{b,Rd}$ is calculated as

$$M_{b,Rd} = \chi_{LT} W_{pl,y} \frac{f_y}{\gamma_{M1}} = 814 \times 3055 \times 355 = 882,803 \text{ Nm} = 883\text{kNm}$$
Torsional Parameters:

Torsional Bending constant, \( a \), is given by

\[
a = \left( \frac{E_{lt} I_{lt}}{G_{lt}} \right)^{0.5} = \left( \frac{210 \times (1,500,000 \times 10^6)}{81 \times (1177 \times 10^4)} \right)^{0.5} = 575 \text{ mm}
\]

\( L/a = 9000/575 = 15.7 \)

Using \( L/a \) value and Table 4.1 of SCI P342, by linear interpolation:

\[
\frac{\phi G_{lt} T_{a}}{T_{a} a} = 1.9 \quad \rightarrow \quad \phi = 0.0124 \text{ radians}
\]

and

\[
- \frac{\phi G_{lt} T_{a}}{T_{a} a} = 0.0634 \quad \rightarrow \quad \phi'' = -1.25 \times 10^{-9} \text{ radians/mm}^2
\]

Interaction of top flange is critical due to beam asymmetry

Minor axis bending moment in top flange, \( M_{zT} \):

\[
M_{zT} = M_{t} \phi = 128 \times 0.0124 = 1.59 \text{ kNm}
\]

Minor axis bending stress in top flange, \( \sigma_{bzT} \):

\[
\sigma_{bzT} = \frac{M_{zT}}{I_{zT} \phi} = \frac{1.59 \times 10^3}{(10463 \times 10^{-5}) \phi} = 1,390,471 \text{ Pa} = 1.39 \text{ MPa}
\]

Distance from centre of gravity to shear centre, \( y_0 \):

\[
y_0 = \frac{h_1}{I_{zc}} - h_{t} l_{zc} = \frac{198 - \frac{40}{2}}{178} = 178 \text{ mm}
\]

\[
h_t = y_e - \frac{t_f}{2} = 198 - \frac{40}{2} = 124 \text{ mm}
\]

\[
h_b = D - h_{t} - t_f = 342 - 178 - 40 = 124 \text{ mm}
\]

\[
l_{zc} = \frac{t_f B_c^3}{12} = \frac{40 \times 183^3}{12} \times 10^{-4} = 2043 \text{ cm}^4
\]

\[
l_{zt} = \frac{t_f B_h^3}{12} = \frac{40 \times 293^3}{12} \times 10^{-4} = 8385 \text{ cm}^4
\]

Hence:

\[
y_0 = \frac{(124 \times 8385) - (178 \times 2043)}{8385 + 2043} = 64.84 \text{ mm}
\]

\[
h_1 = h_t + y_0 = 242.84 \text{ mm}
\]

Normalised Warping function, \( W_{no} \) is given by
Appendix A

XLT slab in 9m x 9m grid Office

W_o = \frac{B_1 h_1}{2} = \frac{183 \times 242.84}{2} = 22,219 \text{ mm}^2

Warping stress, \( \sigma_w \) can be calculated as

\[
\sigma_w = E.W_{no} |\phi''| = (210 \times 10^6) \times 22,219 \times (1.25 \times 10^{-3}) = 5,832,487 \text{ Pa} = 5.83 \text{ MPa}
\]

Equivalent uniform moment factor, \( m_{LT} = 0.925 \) for a non-destabilizing UDL

Finally, putting values into interaction formula for buckling:

\[
\frac{128 \times 0.925}{883} + \left\{ \frac{1.39 + 5.83}{355} \right\} \left\{ 1 + \frac{0.5 \times 128 \times 0.925}{883} \right\} = 0.156 \div \text{OK}
\]

*Check Local Capacity of compression flange in combined bending and torsion*

\[
\sigma_{by} + \sigma_{btT} + \sigma_W \leq f_y \quad (= 355 \text{ MPa})
\]

\[
\sigma_{by} = \frac{M_y}{W_{el,y}} = \frac{128 \times 10^3}{2320} = 55.2 \text{ MPa}
\]

\[
\sigma_{by} + \sigma_{btT} + \sigma_W = 55.2 + 1.39 + 5.83 = 62.4 \text{ MPa} \quad \text{OK}
\]

Construction Load Case 1 is OK for buckling and local capacity of compression flange

*Case 2: XLT slab on both sides, construction loading on one side of ASB only.*

**Construction Loading**

Assume bearing length of 40mm for design

Imposed UDL, \( q_k \) = 0.5 \times 4.5m = 2.25 \text{ kN/m}

Permanent Load (slab) = Slab unit weight = 4.7 \times (260/1000) = 1.22 \text{ kN/m}^2

Permanent Load (beam) = = 1.92 \text{ kN/m}

Permanent UDL, \( g_k \) = slab + beam = (1.22 \times 9m) + 1.92 = 12.9 \text{ kN/m}
### Lateral Torsional Buckling (LTB) for combined bending and tension

**Verify against interaction formula:**

\[
\frac{M_y m_{LT}}{M_{b, Rd}} + \left( \frac{\sigma_{bLT} + \sigma_w}{f_y} \right) \left( 1 + \frac{0.5 \cdot M_y m_{LT}}{M_{b, Rd}} \right) \leq 1
\]

**Major axis moment, \( M_y \):**

\[
\frac{f_d t^2}{8} = \frac{19.8 \times 9^2}{8} = 200 \text{ kNm}
\]

**Torsion Load:**

\[
4.5m \times 9m \times (1.5 \times 0.5) = 30.4 \text{ kN}
\]

**Applied torsion, \( T_q \):**

\[
30.4 \times (126.5/10^3) = 3.85 \text{ kNm}
\]

**Torsion Parameters:**

As before,

\[
\frac{\varphi G I_T}{T_q a} = 1.9
\]

\[
-\left( \frac{\varphi^\prime G I_T}{T_q} \right) = 0.0634
\]

Under new torsion loads and value of \( T_{q\varphi} \),

\[
\varphi = 4.4 \times 10^{-3} \text{ radians}
\]

\[
\varphi^\prime = -4.45 \times 10^{-10} \text{ radians/mm}^2
\]

**Minor axis bending moment in top flange, \( M_{zT} \):**

\[
M_{zT} = M_y \varphi = 200 \times 4.4 \times 10^{-3} = 0.88 \text{ kNm}
\]

**Minor axis bending stress in top flange, \( \sigma_{bzT} \):**

\[
\sigma_{bzT} = \frac{M_{zT}}{I_z} = \frac{0.88 \times 10^3}{(10463 \times 10^{-5})} = 769,569 \text{ Pa} = 0.77 \text{ MPa}
\]
Appendix A

XLT slab in 9m x 9m grid Office

Warping stress, $\sigma_w$:

$$\sigma_w = E.W_{no} |\varphi''| = (210 \times 10^3) \times 22,219 \times (4.45 \times 10^{10}) = 2,076,365 \text{ Pa} = 2.08 \text{ MPa}$$

Interaction Formula for buckling:

$$\frac{200 \times 0.925}{883} + \left\{ \frac{0.77 + 2.08}{355} \right\} \left\{ 1 + \frac{0.5 \times 200 \times 0.925}{883} \right\} = 0.218 \div OK$$

*Check Local Capacity of compression flange in combined bending and torsion*

$$\sigma_{by} + \sigma_{btz} + \sigma_W \leq f_y \ (= 355 \text{ MPa})$$

$$\sigma_{by} = \frac{M_y}{W_{el,y}} = \frac{200 \times 10^3}{2320} = 86.2 \text{ MPa}$$

$$\sigma_{by} + \sigma_{btz} + \sigma_W = 86.2 + 0.77 + 2.08 = 89.1 \text{ MPa} \quad \text{OK}$$

Construction Load Case 2 is OK for buckling and local capacity of compression flange

Construction Load case 3 is not critical as no torsion due to assumption of equal slab spans either side of ASB.

**A.4.3 Normal Stage**

Imposed Load = Office Loading = 2.5 kN/m$^2$

*Case 1: Pattern Loading, super-permanent loading on both sides, imposed loading on one side of ASB only.*

[Diagram showing imposed and superimposed loads]

Assume bearing length of 40mm for design

Imposed UDL, $q_k$ = 2.5 x 4.5m = 11.25 kN/m

Permanent Load (slab) = Slab unit weight = 4.7 x (260/1000) = 1.22 kN/m$^2$

Permanent Load (beam) = 1.92 kN/m

Permanent UDL, $g_k$ = (slab + superimposed) + beam = (1.22+0.5)x9m + 1.92 = 17.4 kN/m
Lateral Torsional Buckling (LTB) for combined bending and tension

Verify against interaction formula:

\[ \frac{M_y m_{LT}}{M_{b,Rd}} + \left( \frac{\sigma_{bLT} + \sigma_w}{f_y} \right) \left( 1 + \frac{0.5 \cdot M_y m_{LT}}{M_{b,Rd}} \right) \leq 1 \]

**Major axis moment, \( M_y \):**

\[ \frac{f_d t^2}{8} = \frac{38.6 \times 9^2}{8} = 391 \text{ kNm} \]

**Torsion Load:**

\[ 4.5 \times 9 \times (1.5 \times 2.5) = 152 \text{ kN} \]

**Applied torsion, \( T_q \):**

\[ 152 \times (126.5/10^3) = 19.2 \text{ kNm} \]

**Torsion Parameters:**

As before,

\[ \frac{\varphi G I_y}{T_q a} = 1.9 \]

\[ -\left( \frac{\varphi'' G I_y a}{T_q} \right) = 0.0634 \]

Under new torsion loads and value of \( T_{aq} \):

\[ \varphi = 0.022 \text{ radians} \]

\[ \varphi'' = -2.22 \times 10^{-9} \text{ radians/mm}^2 \]

**Minor axis bending moment in top flange, \( M_{zT} \):**

\[ M_{zT} = M_y \varphi = 391 \times 0.022 = 8.60 \text{ kNm} \]

**Minor axis bending stress in top flange, \( \sigma_{bzT} \):**

\[ \sigma_{bzT} = \frac{M_{zT}}{I_z / 0.5H} = \frac{8.6 \times 10^3}{(10463 \times 10^{-5}) / (0.5 \times 183)} = 7,520,788 \text{ Pa} = 7.52 \text{ MPa} \]
Warping stress, $\sigma_w$:

$$\sigma_w = E.W_n.\varphi'' = (210\times10^9) \times 22,219 \times (2.22 \times 10^{-9}) = 10,358,498 \text{ Pa} = 10.4 \text{ MPa}$$

Interaction Formula for buckling:

$$\frac{391 \times 0.925}{883} + \left( \frac{7.52 + 10.4}{355} \right) \left( 1 + \frac{0.5 \times 391 \times 0.925}{883} \right) = 0.47 \therefore \text{OK}$$

Check Local Capacity of compression flange in combined bending and torsion

$$\sigma_{by} + \sigma_{btT} + \sigma_W \leq f_y \ (= 355 \text{ MPa})$$

$$\sigma_{by} = \frac{M_y}{W_{el,y}} = \frac{391 \times 10^3}{2320} = 169 \text{ MPa}$$

$$\sigma_{by} + \sigma_{btT} + \sigma_W = 169 + 10.4 + 7.52 = 187 \text{ MPa} \quad \text{OK}$$

Normal Stage pattern loading is OK for buckling and local capacity of compression flange

Case 2: Maximum loading

Imposed Load

Superimposed Dead Load

For this load case, need only check against buckling capacity and shear failure

Imposed UDL, $q_k = 2.5 \times 9\text{m} = 22.5 \text{ kN/m}$

Permanent Load (slab) = Slab unit weight = 4.7 x (260/1000) = 1.22 kN/m²

Permanent Load (beam) = 1.92 kN/m

Permanent UDL, $g_k = \text{(slab + superimposed) + beam} = (1.22+0.5) \times 9\text{m} + 1.92 = 17.4 \text{ kN/m}$

$$f_{d,1} = \gamma_G.g_k + \gamma_{Q1}.\Psi_0.q_k = (1.35 \times 17.4) + (1.5 \times 0.7 \times 22.5) = 47.1 \text{ kN/m}$$

$$f_{d,2} = \xi \gamma_G.g_k + \gamma_{Q1}.q_k = (0.925 \times 1.35 \times 17.4) + (1.5 \times 22.5) = 55.5 \text{ kN/m}$$

$$f_d = \text{max}(f_{d,1}, f_{d,2}) = 55.5 \text{ kN/m}$$
Major axis moment, $M_y = \frac{f_d l^2}{8} = \frac{55.5 \times 9^2}{8} = 562$ kNm

**Buckling Capacity**

\[
\frac{M_y}{M_{p,Rd}} \leq 1.0 : \quad \frac{562}{883} = 0.636 \therefore OK
\]

**Shear Capacity**

Design shear force, $V_d = \frac{f_d \times l}{2} = \frac{55.5 \times 9}{2} = 250$ kN

Design shear resistance $V_{pl,Rd} = A_v \left( \frac{f_y}{\sqrt{3}} \right)_{\gamma_M}

A_v = A - (B_b t_t + B_t t_c) + (t_w + 2r)t_c = (249 \times 100) - (293 \times 40 + 183 \times 40) + (20 + 2 \times 27) \times 40 = 8820 \text{mm}^2

\[
V_{pl,Rd} = 8820 \times \left( \frac{355}{\sqrt{3}} \right)_{1.0} = 1,807,741 \text{ N} = 1,807 \text{ kN}
\]

\[
\frac{V_d}{V_{pl,Rd}} \leq 1.0 : \quad \frac{250}{1807} = 0.138 \therefore OK
\]

**A.4.4 Serviceability Checks**

Composite behaviour of the timber slab and steel beam is assumed for deflection checks at the normal loading stage. The 2nd moment of area of the composite beam section must be found for use in the calculations.

**Effective Breadth**

When using concrete slabs, with infill concrete around the web of the beam, effective breadth is assumed as span/32 to either side of the beam. To replicate this for the timber case, without infill, the effective breadth, $b_{eff}$ is taken as

\[
b_{eff} = \frac{\text{span}}{16} - (B_b - 160 \text{mm}) = \frac{9000}{16} - (293 - 160) = 430 \text{ mm}
\]

This accounts for the area between the slab and the ASB web being empty.
Appendix A

XLT slab in 9m x 9m grid Office

Modular Ratio

The modular ratio, \( n \), is a ratio of the Young’s Moduli of the two materials in the composite section and is used to make their different properties compatible in calculations. By using the modular ratio it is possible to transform the cross-section area of one material into an equivalent area of the other material. This additional equivalent area is used to calculate the enhancement in behaviour of the two materials acting compositely versus the one material acting alone.

To transform the timber slab into an equivalent area of steel, the modular ratio is given by,

\[
  n = \frac{E_{\text{steel}}}{E_{\text{timber}}}
\]

Due to its susceptibility to creep, the deflection of timber elements normally consists of two parts – an instantaneous deflection, and an additional long term deformation. However, due to the presence of a substantially stiffer element, the ASB, and the use of composite theory, it is thought sufficient to account for loss of stiffness of the timber over time when calculating the Young’s Modulus. As such, \( E_{\text{timber}} = E_{d,SLs} = \frac{E_{\text{mean}}}{(1+k_{\text{def}})} = \frac{12 \text{ GPa}}{1+0.8} = 6.67 \text{ GPa} \)

\[
  n = \frac{210}{6.67} = 31.5
\]

1st Moment of Area to find composite neutral axis depth, \( y_{\text{comp}} \)

About the top of the slab, in steel units...

\[
  \left[ \left( \frac{b_{\text{eff}} \cdot d}{n} \right) + A_{\text{ASB}} \right] \cdot y_{\text{comp}} = \left( \frac{b_{\text{eff}} \cdot d}{n} \cdot \frac{d}{2} \right) + (A_{\text{ASB}} \cdot d_{\text{NA}})
\]

Where \( A_{\text{ASB}} \) is the cross sectional area of the ASB, and \( d_{\text{NA}} \) is the distance between the top of the slab and the neutral axis of the ASB, and is given by:

\[
  d_{\text{NA}} = y_c - (D - t_f - d) = 198 - (342 - 40 - 260) = 156 \text{ mm}
\]

\[
  \left[ \left( \frac{430 \times 260}{31.5} \right) + (249 \times 10^2) \right] \cdot y_{\text{comp}} = \left( \frac{430 \times 260^2}{2 \times 31.5} \right) + (249 \times 10^2) \times 156
\]

\[
  \Rightarrow y_{\text{comp}} = 152.8 \text{ mm}
\]

Composite 2nd Moment of Area, using Parallel Axis Theorem

In steel units...

\[
  I_{\text{comp}} = \left[ \frac{b_{\text{eff}} \cdot d^3}{12n} + \left( \frac{b_{\text{eff}} \cdot d}{n} \times \left( y_{\text{comp}} - \frac{d}{2} \right)^2 \right) \right] + \left[ I_{\text{ASB}} + A_{\text{ASB}} \cdot (d_{\text{NA}} - y_{\text{comp}})^2 \right]
\]

Where \( I_{\text{ASB}} \) is the 2nd moment of area for the ASB. Substituting values into the formula,

\[
  \Rightarrow I_{\text{comp}} = 48,078 \text{ cm}^4
\]

Construction stage deflection check (bare steel properties)
Self weight deflection (slab on both sides):

$$\omega_{SLS} = \text{slab + beam} = (1.22 \times 9) + 1.92 = 12.9 \text{ kN/m}$$

Deflection during construction, $\delta_c$, given by:

$$\delta_c = \frac{5}{384} \frac{\omega_{SLS} L^4}{EI} = \left(\frac{5}{384}\right) \times \frac{12.9 \times 10^3 \times 9^4}{210 \times 45.871 \times 10} = 0.0114 \text{ m} = 11.4 \text{ mm}$$

Construction Deflection limit = span/200 = 9000/200 = 45 mm OK

Normal stage deflection check (composite properties)

Imposed load deflection check

$$\omega_{SLS} = \text{imposed load} = 2.5 \times 9 \text{ m} = 22.5 \text{ kN/m}$$

$$\delta_{IMP} = \frac{5}{384} \frac{\omega_{SLS} L^4}{EI} = \left(\frac{5}{384}\right) \times \frac{22.5 \times 10^3 \times 9^4}{210 \times 48.078 \times 10} = 0.0190 \text{ m} = 19.0 \text{ mm}$$

Imposed Deflection limit = span/360 = 9000/360 = 25 mm OK

Super-imposed permanent load deflection check

$$\omega_{SLS} = \text{superimposed} = 0.5 \times 9 \text{ m} = 4.5 \text{ kN/m}$$

$$\delta_{SUP} = \frac{5}{384} \frac{\omega_{SLS} L^4}{EI} = \left(\frac{5}{384}\right) \times \frac{4.5 \times 10^3 \times 9^4}{210 \times 48.078 \times 10} = 0.00381 \text{ m} = 3.81 \text{ mm}$$

As deflections arising during construction are corrected on site, total deflection consists solely of those due to imposed load and superimposed permanent load:

Total deflection = $\delta_{IMP} + \delta_{SUP}$ = 19.0 + 3.81 = 22.8 mm

Check: 22.8 < span/200 (= 25mm) Hence OK

300 ASB 196 is verified for this situation and loading
A.4.5 Tie Beam

Assuming an internal bay,

Design Tie force, \( T_i = 0.8(g_k + \psi_1q_k)sL \)

Where

\( s \) is the tie spacing \( = 9 \text{ m} \)

\( L \) is the tie span \( = 9 \text{ m} \)

\( \psi_1 \) \( = 0.5 \)

\( g_k \) slab weight + superimposed \( = 1.72 \text{ kN/m}^2 \)

\( q_k \) imposed load \( = 2.5 \text{ kN/m}^2 \)

Substituting, \( T_i = 192 \text{ kN} \)

Minimum section area required \( = \frac{T_i}{f_y} = \frac{192}{355} \times 10 = 5.41 \text{ cm}^2 \)

To avoid visible sag, min. section depth \( = \frac{\text{span}}{40} = \frac{9000}{40} = 225\text{mm} \)

To meet requirements, use 165x267x33 T-section as structural tie.