Improved Structural, Embodied Carbon and Cost Efficiency of Single Storey Industrial Buildings using Sandwich Panels

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To my grandparents

Dimitris and Toula

Michalakis and Kalliopi

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Keywords

Sandwich panels, Composite panels, Profiled insulated panels, Steel structures, Industrial buildings, Single storey buildings, Building envelope, Cladding, Insulation, Polyisocyanurate (PIR), Portal frames, Trusses, Northlights, Rooflights, Long span, Diaphragm action, Stressed skin, Frameless buildings, Steelwork reduction, Embodied carbon, Low carbon, Construction cost, Optimisation, Structural testing, Sustainability

Synopsis

Recent changes to the Building Regulations aimed at improving energy efficiency have resulted in significant increases in the amount of insulation incorporated into building envelopes. As a consequence, composite (sandwich) panels have become deeper, considerably improving their structural capability in terms of strength and stiffness. This however has largely been ignored in the design of building structures, so this study has sought to ascertain the degree to which more efficient solutions, that take advantage of the improved capabilities of panels, may reduce the embodied carbon of building structures, and indeed of whole buildings.

The research focused on single-storey industrial buildings. A series of studies were undertaken to evaluate the opportunities, and to quantify the benefits and trade-offs associated with structural solutions that fully exploit panel capabilities. The studies addressed a) long span sandwich panels to reduce the number of supporting structural members, b) diaphragm action to stiffen the frame and c) frameless buildings. Results suggested that the greatest potential benefit (up to 60% steelwork saving) arises from the use of long span systems, particularly for trussed roof frames with northlight construction.

The study identified that further realistically achievable improvement in the spanning capabilities of panels would provide significant additional benefits. An improved long span sandwich panel design was therefore developed using theoretical investigations, structural testing and a Pareto-optimisation process. The optimal solution in terms of embodied carbon and panel strength was defined.

Optimised frame arrangements combined with enhanced long span roof sandwich panels were compared with traditional portal frame solutions. This comparison demonstrated the considerable savings in terms of both embodied carbon and cost that can be achieved over traditional construction. The study demonstrated that exploiting the increased insulation depth of composite panels can deliver solutions with greater structural efficiency and reduced environmental impact.

Publications

Parts of this thesis have been published by the author in the papers listed below.

International Refereed Conference Papers

Moutaftsis, D., Heywood, M.D., Ogden, R.G. (2015) 'Improved Structural Efficiency in Steel Single Storey Buildings using Sandwich Panels', *Proceedings of the 8th International Conference on Advances in Steel Structures*. 21-24 July. Lisbon, Portugal.

Moutaftsis, D., Resalati, S., Heywood, M.D., Ogden, R.G. (2015) 'Optimised Low Embodied and Operational Carbon Solutions for Single Storey Industrial Buildings'. *Proceedings of the Eight International Structural Engineering and Construction Conference (ISEC-8): Implementing Innovative Ideas in Structural Engineering and Management.* 23-28 November. Sydney, Australia.

Heywood, M.D., Moutaftsis, D., Ogden, R.G. (2016) 'Embodied carbon efficient industrial buildings with optimised long span sandwich panels'. *Proceedings of the First European and Mediterranean Structural Engineering and Construction Conference (EURO-MED-SEC-1): Interaction between Theory and Practice in Civil Engineering and Construction.* 24-29 May. Istanbul, Turkey.

Non-Refereed Conference Papers

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Oral presentations

In addition, parts of this thesis have been orally presented by the author in the following conference:

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Chapter 1 Introduction

1.1 Background

1.1.1 Climate change targets

The increase in greenhouse gas emissions into the Earth's atmosphere is the major contributor to global warming and the extension of greenhouse effect beyond natural levels (United Nations, 1992). Greenhouse gas emissions consist primarily of water vapour, carbon dioxide, methane, nitrous oxide and ozone. Among all these gases, water vapour and carbon dioxide have the most direct contribution to the greenhouse effect (Kiehl and Trenberth, 1997). Since the energy demand in the modern world is largely met by burning fossil fuels (86% according to the World Bank), carbon dioxide is directly related to energy consumption. The growing realisation of the link between global warming and fossil fuels consumption (Raupach *et al*., 2007, Denman *et al*., 2007) resulted in global efforts to significantly reduce greenhouse gas emissions.

The United Nations Framework Convention on Climate Change (UNFCCC) is the most important international influence on the European and consequently the United Kingdom (UK) energy consumption policies (Ekins and Lees, 2008). The Kyoto Protocol (United Nations, 1998) was agreed in 1997 under the UNFCCC and came in force in 2005. It comprised the binding commitment of industrialised countries to reduce their greenhouse gas emissions. The initial target was a reduction of the greenhouse gases by an average of 5% during 2008 – 2012 in comparison to 1990 levels. The Protocol also recognised that 'developed' countries were primarily responsible for the current high levels of greenhouse gas emissions due to their heavy industrial activities over the past two centuries; therefore, a heavier greenhouse gas reduction burden was assigned to 'developed' nations (United Nations, 1998).

The European Union (EU)'s response to the Kyoto Protocol's binding commitment was the adoption of the 'Energy Policy for Europe' plan by the Commission of the European Communities (CEC, 2007). The plan called for 20% energy saving, 20% greenhouse gas emissions reduction, 20% of the overall Union's energy demand to be covered by renewable sources and, finally, 10% of the energy demand in transportation to be covered by renewable measures, all by 2020 (CEC, 2007).

The UK Climate Change Act 2008 (The Stationary Office, 2008) issued by the UK Government has set a legally binding target for a reduction of greenhouse gases of at least 80% by 2050 with an intermediate target of 36% by 2020 in comparison with 1990 levels. These targets are more onerous and challenging compared to the EU target of 20% overall carbon reduction by 2020.

1.1.2 Building carbon emissions

In developed countries, buildings are a major source of energy consumption, representing levels between 20% and 40% of the national energy demand (Pérez-Lombard *et al*., 2008). Population growth, improvement of living standards and comfort demand and increase of time spent inside the buildings were identified as the primary reasons of the increasing energy consumption (and consequently carbon emissions). In the European Union (EU) and the USA, such consumption has subsequently exceeded the demand of energy-consuming sectors such as transportation and industry. According to the Buildings Energy Data Book (2011), published by the US Department of Energy, buildings in Europe are responsible for 40% of energy consumption and 36% of carbon emissions. The World Business Council for Sustainable Development published that buildings are responsible for more than 40% of the energy demand in most nations (WBCSD, 2007). Pérez-Lombard *et al*. (2008) stated that 'energy efficiency in buildings is today a prime objective for energy policy at regional, national and international levels'.

The EU has recognised that the energy efficiency of buildings is a key objective of its energy and climate policies, as stated within the Action Plan for Energy Efficiency (CEC, 2006). The main regulatory means towards achieving the EU energy efficiency requirements in the building sector in line with the obligations against the Kyoto Protocol is the Energy Performance of Buildings Directive (EPBD) (European Parliament and Council, 2002), adopted in 2002 and put into force in 2006. While buildings are different across Europe, the directive introduces a generic methodology framework to measure energy efficiency and allows each Member State to adopt a regulation at a national or regional level based on that generic framework. Ekins and Lees (2008) have considered that the EPBD is the main European regulation which impacts the UK built environment.

Buildings in the UK account for the 39% of the overall energy consumptions (Pérez-Lombard *et al*., 2008) and are responsible for the 50% of the carbon emissions nationwide (BRE, 2006), rates which are higher than the EU average. Therefore, significant improvements in the energy performance of the buildings have been and continue to be required in order to meet the specified carbon reduction targets.

The UK Government has previously set a target for all new domestic buildings to be 'Zero Carbon' from 2016 and new non-domestic buildings from 2019 (DCLG, 2007). Although the UK government has recently put these deadlines on hold on economic grounds (HM Treasury, 2015), there is still a requirement that the UK abides by its legally binding targets to reduce carbon emissions. Given the dominating contribution of buildings in the UK to the country's operational carbon emissions, it is almost certain that achievement of 'Zero Carbon' buildings will inevitably be required to be achieved in the future. The definition of 'Zero Carbon' still remains under consultation (UK Green Building Council). However, as a minimum, it will demand that 100% of their operational energy and consequential carbon emissions regulated under Part L of the Building Regulations for Conservation of Fuel and Energy to be provided by renewables (Zero Carbon Hub). As 'regulated' energy and emissions are classified those associated with the building fabric and fixed services and excluding cooking, appliances and IT equipment. According to Sansom and Pope (2012), reducing the operational carbon emissions associated with buildings is the primary sustainable construction driver in the UK.

On the trajectory of achieving the challenging targets set by the UK Climate Change Act and 'Zero Carbon', Building Regulations concerning energy efficiency in England and Wales have been revised in 2002, 2006, 2010 and 2013, setting more demanding energy efficiency standards for buildings in every new revision. The focus of the Building Regulations has been on reducing the operational energy and consequential carbon associated with the building use. For that purpose, measures to reduce operational carbon have been adopted, such as increased insulation thickness in the building envelope, provisions for energy efficient lighting, heat recovery and renewable technologies. However, such measures have resulted in higher levels of material usage in buildings. Consequently, as the operational carbon efficiency increases, so does the relative importance of the embodied carbon (Sansom and Pope, 2012). This trend looks set to continue. In the 1980s it was estimated that the operational energy requirement of buildings was approximately ten times greater than the embodied energy. However, as energy efficiency standards for new buildings have become more onerous, this ratio has shifted to the point where there is far greater parity between operational and embodied energy. Resalati (2015) highlights that considering energy efficiency in terms of operational carbon alone is no longer justifiable and that there is a compelling logic that the total operational and embodied carbon should both be considered when assessing the carbon performance of buildings.

There is an increased recognition of the significance of embodied carbon in the future carbon reduction studies by many researchers and recommendations by government organisations (HM Government, 2010, UK Green Building Council) that 'whole life cycle' approaches should be adopted in the future regulatory context. Those approaches would require incorporating full aspects of energy and carbon, including in-use and those associated with extraction, processing, manufacture and transportation of the materials, as well as construction and end-of-life processes.

1.2 Problem definition

The trend of the UK Building Regulations to date has been to specify continuously increasing insulation thicknesses for the building envelope to minimise energy losses, resulting in higher levels of embodied carbon for the building as a result of increased material usage. Despite strong evidence of diminishing marginal returns in terms of operational energy savings coupled with increases in the embodied carbon of the insulated material (Resalati, 2015), this trend looks set to continue.

Historically, insulation was considered an 'add in' material, the sole purpose of which was to minimise conductive heat losses. However, for envelope systems acting compositely, the structural capability of the envelope that arises from the thicker insulation is not currently fully exploited. Highly insulated building envelopes possess structural capability in terms of strength and stiffness that could be utilised to minimise materials usage and reduce materials-related embodied carbon. Therefore, an opportunity exists to maximise the utility of the envelope to reduce the overall amount of material and consequential embodied carbon within the whole building.

1.2.1 Sandwich panels

Sandwich panels, also known as composite insulated panels, are prime examples of envelope systems whose structural capacity benefits from increased insulation thickness. Such assemblies are excellent examples of lightweight composite construction, comprising a rigid layer of insulation (typically referred to as 'core') sandwiched between, and bonded to, two thin layers of metal sheeting (typically referred to as 'faces') to form a single manufactured unit. An example is shown in Figure 1.1. The system benefits from:

- high strength to weight ratio
- good structural and thermal performance

- features for air-tight fastening
- excellent sound insulation
- ease of handling and speedy installation
- high speed of production up to 12m/min
- increased durability and reusability
- good options for aesthetics.

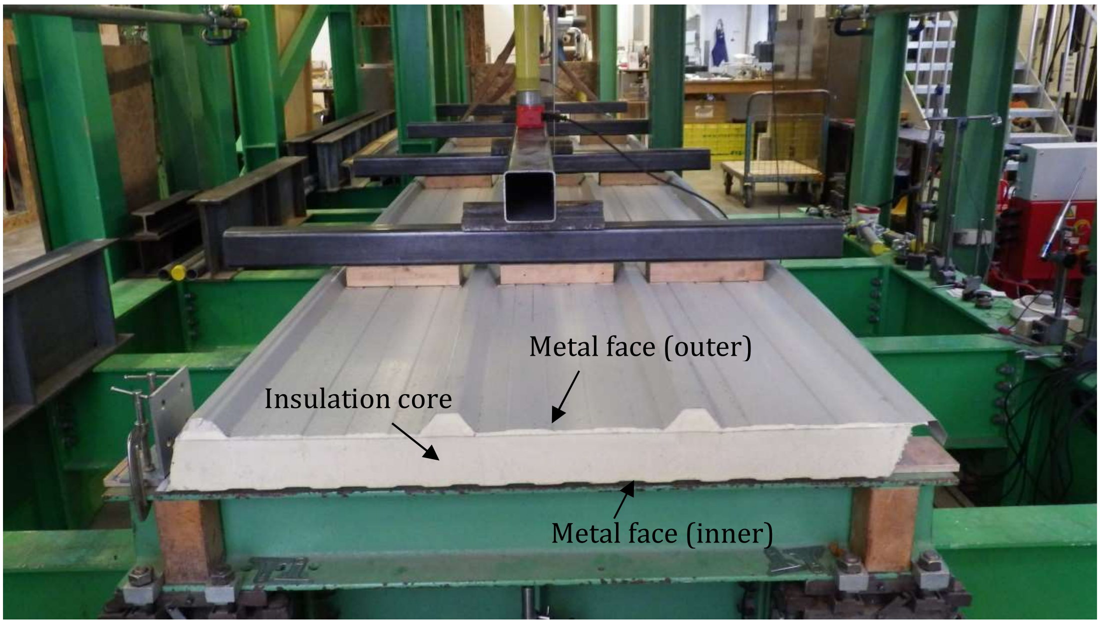


Figure 1.1 Sandwich panel

Sandwich panels are well-established roof and wall envelope systems. They are particularly popular for industrial and office applications, while there has been a growing use in the residential sector during the recent years. According to EPIC (2006), metal-faced sandwich panels accounted for approximately 60% of the building envelope market share for new non-residential buildings at the end of 2006, while the remaining percentage was largely met by metal built-up systems. Furthermore, sandwich panels are popular solutions for internal wall and ceiling applications.

The metal faces are typically made of steel or, occasionally, of aluminium with a thickness range between 0.3mm and 0.7mm. Their geometry can be flat, micro-ribbed (often referred to as 'satinlined'), lightly profiled or fully profiled. Roof applications typically comprise a fully profiled external layer and a lightly profiled internal (liner) layer, while wall applications comprise mostly flat, lightly profiled or micro-ribbed geometries. The core is typically made of Polyisocyanurate (PIR), Polyurethane (PUR), Extruded Polysterene (EPS) or Mineral Wool (slabs or lamellas). In the UK, PIR and Mineral Wool are the most extensively used sandwich panel core materials, with the latter ones being used primarily for wall applications. Typical cross-sections of sandwich panels are shown in Figure 1.2.

a)

b)

c)

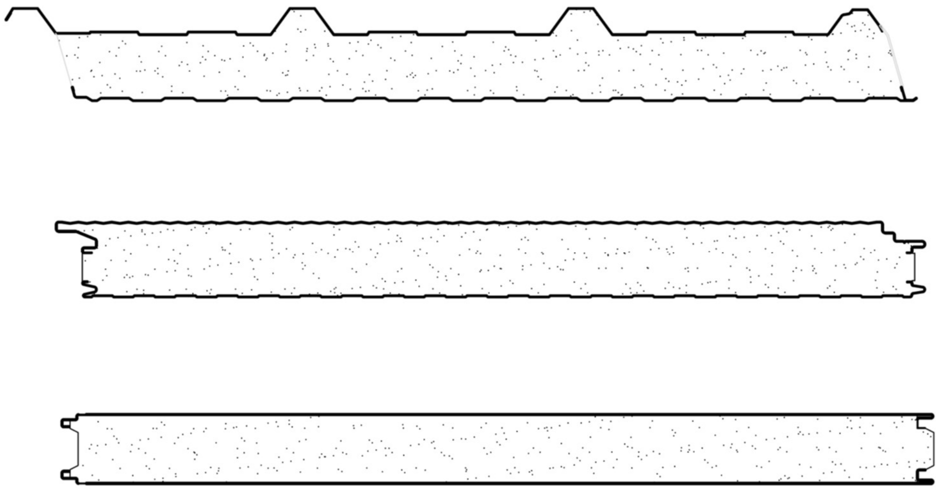


Figure 1.2 Typical sandwich panel cross-sections with: a) fully profiled faces; b) lightly profiled faces; and c) flat faces

The behaviour of a sandwich panel relies on the composite action between the insulation core and the bonded metal faces. Each of the three layers has a significant role in the response of the panels under structural and thermal loads and these are later discussed in detail. One of the major functions of the core, besides its insulating ability, is the fact that it keeps the metal faces apart and increases their moment of inertia, enabling the assembly to act as a single beam. Furthermore, it increases the strength of the metal faces by providing a stabilising function against local failure whilst providing shear connection between them. As a result, the increase of the insulation depth in a sandwich panel leads to an increase of the component's stiffness, resistance and, consequently, bending moment capacity, in a similar manner to a beam with a deeper web. The insulation depth is the most dominant factor, by far, in determining the sandwich panel resistance to bending. In theory, this raises the potential for increasing the span of sandwich panel elements, although there are technical and practical barriers. Therefore, there is scope to:

a) evaluate the opportunity arising from this insulation increase
b) quantify the benefit and trade-off with elimination of structure within the building
c) understand the associated barriers.

1.2.2 Single storey industrial buildings

According to the UK Green Building Council, non-domestic buildings in the UK account for 17% of the carbon emissions at a national level. DCLG (2013) predicts an increase of the UK non-domestic building floor area by 35% by 2050. According to DCLG (2013), 42% of the anticipated mix for new non-domestic buildings used in the latest policy-making studies is warehouses, used for manufacturing, retail and distribution. All these building types in the UK are typically built by the same type of construction which comprises single storey steel frames. The same type of construction is widely used for buildings with other applications, such as leisure, sports and transport. For the purpose of the present study, all the buildings comprising single storey steel frames and used for the wide range of the aforementioned activities are defined as 'single storey industrial buildings', also commonly referred to as 'single storey sheds'. Consequently, carbon emission savings in single storey industrial buildings would have a considerable impact in the non-domestic building sector and at national level.

Furthermore, the single storey industrial building sector attracts almost 60% of the constructional steelwork consumed in the UK (BCSA, 2003, Steelconstruction.info, 2016). Consequently, any structural material savings in this type of buildings will have a significant impact in this market sector. According to Owens (2006), steel frames hold 95% of the market share for this type of buildings in the UK, while similar high rates are evident in other European countries. According to Raven and Heywood (2006), this sector comprised significant annual revenue of £1bn for the frames and £1.5bn for the envelope in 2006.

Single storey building construction in the UK is dominated by steel portal frame structures. Their cost and structural efficiency competitiveness against other materials and forms is primarily due to plastic design, which allows the capacity of each member to be fully utilised with the use of strict restraint requirements. Whilst portal frame construction provides optimised economy and efficiency for the structure itself, it is not necessarily the optimal form for the building as a whole. Alternative structural forms may yield greater benefit when viewed holistically, especially when the structural capability of the envelope is utilised.

1.3 Research aims and objectives

The primary aims of this research are to:

- Quantify the benefit associated with reduction of structure within the building by exploiting the increased insulation depth and structural capability of sandwich panel envelope systems.
- Address the technical barriers to the implementation of more efficient structure-envelope assemblies.

The objectives subordinate to these aims are to:

1. Perform literature search to:
    a) Review the technology and state-of-the-art in the design and construction of single storey industrial buildings in the UK.
    b) Review the UK regulatory context for carbon emissions, the trend of the future likely energy conservation requirements, the environmental performance of single-storey industrial buildings, the role of the building envelope and the related opportunities for embodied carbon emissions reduction.
    c) Identify opportunities for exploiting the structural capabilities of sandwich panels arising from the increased insulation thickness in single story buildings.
    d) Review earlier work and state-of-the-art associated with these opportunities.
2. Determine the opportunities for exploiting the structural capability of sandwich panel envelope systems in terms of:
    a) Increasing the span of the cladding elements (reducing the number of structural members).
    b) Utilising diaphragm action within the envelope (building stability and stiffness).
    c) Removal of primary frame elements (frameless construction for small buildings).
3. Determine the structural forms that are best able to utilise the structural capability of sandwich panels, identify barriers (technical and commercial) to the uptake of these types of structure and propose solutions to overcome these barriers.
4. For the identified technical barriers, carry out analytical and experimental work to:
    a) Propose sandwich panel solutions to accommodate the enhanced structural requirements for the selected building forms through:

- Assessing the impact of material properties reliability and design guidance conservatism on the modelling of structural performance.
- Design of improved sandwich panel solutions, making use of current guidance and test data and through application of design optimisation methods.

b) Evaluate the performance of the systems in terms of embodied carbon.

5. Review the envelope – structural forms assemblies in terms of:
   a) Structural efficiency, based on structural weight.
   b) Embodied carbon, based on established databases and selected system boundaries, reflecting the identified envelope forms and considering the optimum combination of envelope and structure for the chosen buildings.
   c) Cost on site, based on calculated component and construction rates.

1.4 Structure of thesis

The detailed investigations undertaken to address the aims of the research are presented in nine chapters as summarised below:

Chapter 1 introduces the background of the study, the research aims and objectives and the thesis layout.

Chapter 2 presents the literature search undertaken to (a) review the technology and state-of-the-art in the design and construction of single storey industrial buildings in the UK; (b) review the UK regulatory context for carbon emissions, the trend of the future likely energy conservation requirements, the environmental performance of single-storey industrial buildings, the role of the building envelope and the related opportunities for embodied carbon emissions reduction; (c) identify opportunities for exploiting the structural capabilities of sandwich panels arising from the increased insulation thickness in single story buildings; and (d) review earlier work and state-of-the-art associated with these opportunities.

Chapter 3 outlines the research methodology adopted in the course of the study.

Chapter 4 investigates the opportunity to exploit long span roof envelope systems to reduce the number of primary structural members in the building.

Chapter 5 investigates the opportunity to exploit the diaphragm action of the roof envelope to enhance the building stability and stiffness.

Chapter 6 investigates the opportunity to produce frameless buildings by replacing primary frame elements with the envelope.

For the opportunities and structural schemes with the greatest benefit in terms of frame material reduction, **Chapter 7** presents the numerical and experimental research undertaken to design carbon-optimal sandwich panels and quantify their consequent embodied carbon emissions.

For the structural schemes with the greatest benefit in material savings combined with the embodied carbon-optimal sandwich panel specifications, **Chapter 8** holistically reviews the entire building (structure and envelope) in terms of embodied carbon and cost.

Chapter 9 presents the conclusions of the research and provides recommendations for further research.

References are brought together and presented alphabetically by author or authority.

Appendices A to F present supplementary numerical data and information associated with calculations.

Chapter 2 Literature review

2.1 Single-storey industrial buildings

Single-storey industrial buildings, typically referred to as ‘sheds’, are the largest sector for constructional steelwork in the UK, accounting for approximately 60% of constructional steelwork (BCSA, 2003, Steelconstruction.info, 2016). The sector includes industrial buildings and also workshops, distribution centres, warehouses, retail, transport, leisure and sports buildings among other applications. Typical sizes vary from a few hundred squares meters for small workshops to large industrial or distribution warehouses of over a million square meters (BCSA, 2006).

While single-storey industrial buildings fulfil many roles, they have similar arrangements, comprising long span steel frames with metal cladding systems. According to a Construction Markets survey commissioned by BCSA and TATA Steel (2012), steel frames account for 98% of single-storey industrial buildings in the UK, with the remaining percentages shared among precast concrete, in-situ concrete and timber frames.

Given that single-storey industrial buildings in the UK are almost exclusively made of steel and account for significant proportion of steelwork consumption, any improvements in the design to yield frame material savings would be likely to have significant impact on material and, consequently, embodied carbon savings across the sector. An overview of the state-of-the-art for the design and construction of single-storey industrial buildings follows.

2.1.1 Anatomy of a steel framed single-storey industrial building

The anatomy of a typical single-storey industrial building is shown in Figure 2.1 and comprises three main layers as per below (Heywood, 2006):

1. The primary steel frame, comprising columns, rafters or trusses and bracing systems, transferring the applied load to the foundations. Depending on the building size and the structural form used, the frame may be single- or multi-bay.
2. The secondary structure, comprising purlins and side rails for the roof and walls respectively. These members support the cladding, transfer the load to the primary steel frame and provide restraint to the primary frame members.

3. The roof and wall cladding, provide a weathertight building, minimise energy losses through their insulating function and provisions for air-tightness, transfer the load to the secondary steelwork, provide restraint to the secondary steelwork members and include provisions for natural light (such as rooflights or north lights) and recently, for renewable energy sources such as photovoltaic panels.

The building envelope also comprises other architectural parts such as windows, doors, gutters and flashings.

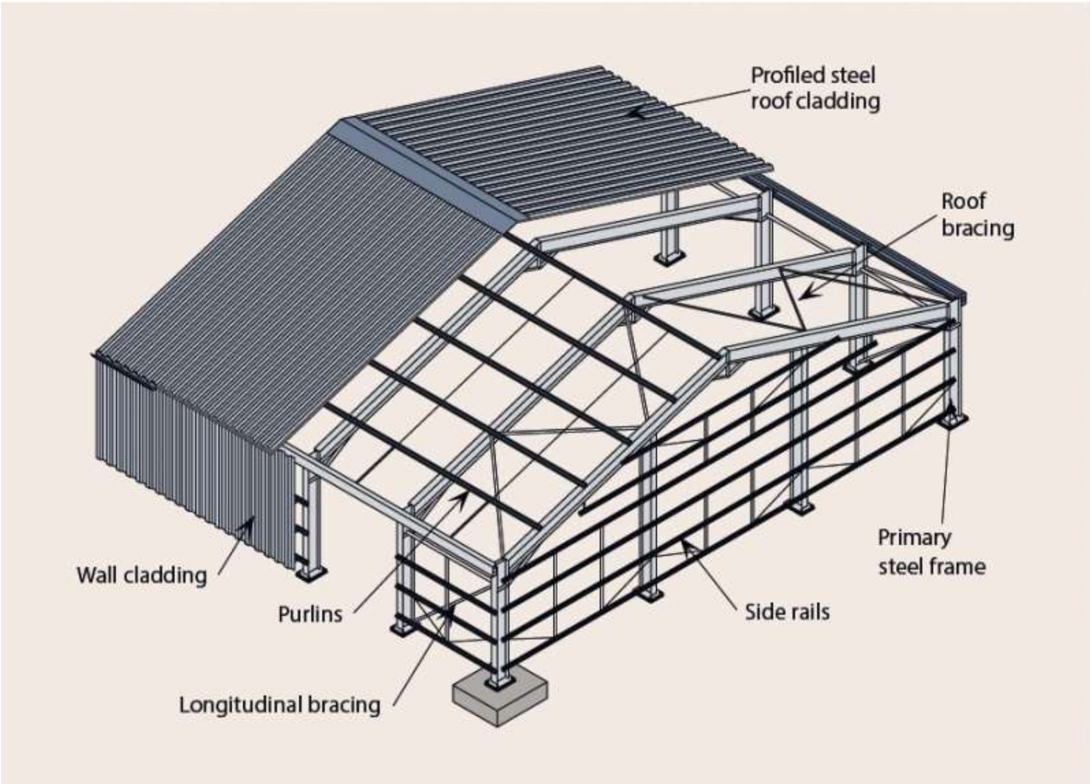


Figure 2.1 Anatomy of a single-storey steel-framed building; Source: Koschmidder and Brown (2012), steelconstruction.info – Image courtesy of the Steel Construction Institute

In addition to the above, a typical base-build single storey building also comprises floor slabs, foundations, mezzanine floors and, typically, a small area (average 5% in plan) of office spaces annexed as a separate building to the main.

The form and anatomy of steel-framed single-storey industrial buildings depends on the function of the building and the specifics of the project. Despite variations among project types and end-uses, most single-storey industrial buildings follow similar construction principles, comprising long-spans and large open plan areas with minimum number of internal columns to allow for flexibility and rearrangements of production lines, plant or equipment throughout the life of the building. These requirements are achieved through the use of long-span steel framing with a weathertight envelope, lately with increased requirements for operational energy efficiency to comply with legislation for energy and carbon reduction. The design of the structural frame and building envelope are highly interconnected and this will be discussed later in greater detail.

The focus of the current research is on the primary frame, secondary steelwork and building envelope (wall and roof cladding). The options for those building elements are described in the following sections.

2.1.2 Primary frame

Three main frame options are typically met across single-storey industrial buildings:

- Portal frames
- Lattice truss systems
- North light truss systems

The main frame options met across single-storey industrial buildings are described herein.

2.1.2.1 Portal frames

An illustration of a typical portal frame cross-section and its restraints is shown in Figure 2.2.

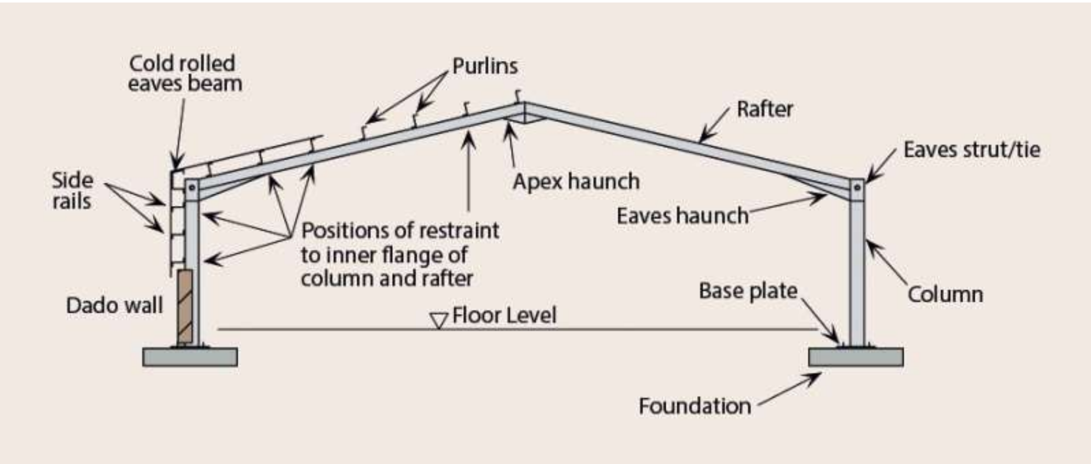


Figure 2.2 Single-span symmetric portal frame with its restraints; Source: Koschmidder and Brown (2012), steelconstruction.info – Image courtesy of the Steel Construction Institute

Portal frames are the predominant form of construction in the single storey industrial building sector and a highly cost-effective solution. They can span up to 60m and can form the most economical solution for the frame particularly at spans up to 35m-40m.

Their high structural and cost efficiency is due to the high interdependence of the frame, secondary structure and building envelope and the efficient, yet onerous, use of restraints (as in Figure 2.2). The use of plastic design has led to highly optimised structures and has flourished in the UK since the 1970s, supported by a reliable procurement framework and the development of bespoke software (Raven and Heywood, 2006).

Portal frames are typically low-rise structures, formed by columns and flat or pitched rafters and achieving large clear spans. The column-rafter connections are moment-resistant and rigidity is achieved with the aid of haunches. The in-plane stability of portal frames is achieved by their in-plane rigidity and the need of bracing is eliminated. Longitudinally, a bracing system is required to provide the lateral stability to the structure. The end frame (gable frame) can be either a portal frame or a braced arrangement of columns and rafters.

Frames are typically constructed with hot-rolled beam and columns sections for the rafters and columns. Cuts from the rafters are typically utilised at the haunch and eaves to achieve robust moment-resisting connections.

Frames can either be single-span or multi-span. When multi-span frames are used, it is quite common that intermediate columns are 'hit-and-miss', i.e. every second intermediate column at valleys is omitted. This provides significant advantage in achieving large column-free areas. Frames are typically spaced between 6m and 8m depending on the building dimensions.

Two methods are used for the design of portal frames, namely elastic and plastic. The elastic design can be performed by conventional software and analytical methods. The plastic analysis requires specialist software which is not always in possession of small or medium design consultancies. The plastic method involves very onerous restraining requirements for the outer and inner flanges of the rafters and columns. It is common that frames are specified based on elastic design and then plastically designed by steelwork fabricators at tender stage.

Portal frames can be designed according to BS EN 1993-1-1:2005 and the relevant UK National Annex or BS 5950-1:200 can also be used. Detailed and state-of-the art guidance is available by Koschmidder and Brown (2012), Brown (2013) and Salter *et. al.* (2004).

2.1.2.2 Lattice truss systems

An illustration of typical lattice truss systems is shown in Figure 2.3.

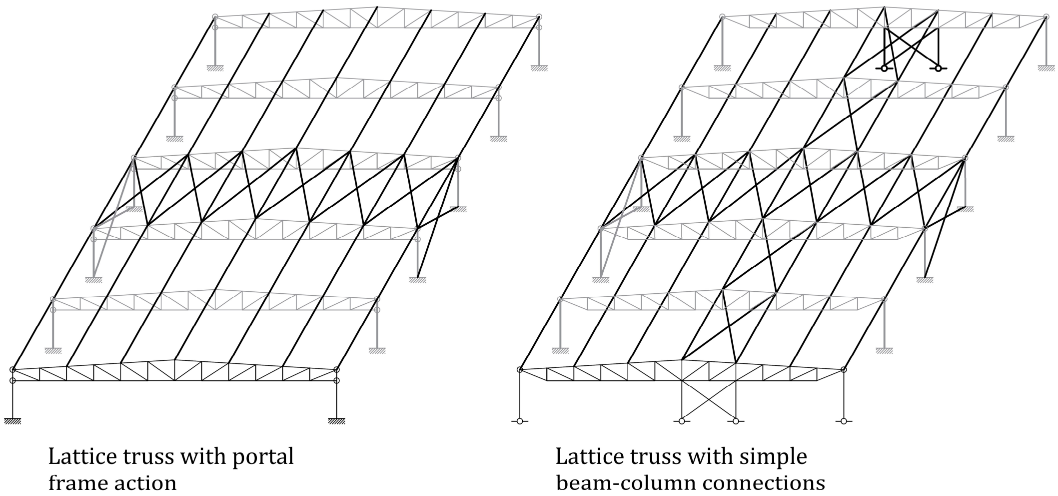


Figure 2.3 General truss arrangements (adapted from steelconstruction.info)

Lattice trusses are well-established structural systems for single-storey industrial buildings and offer the main alternative to portal frame construction. Their unit weight per roof area is often lower than portal frames (Owens, 2012), however the relative cost of fabrication over material is higher than for portal frames which utilise single beam girders. This leads to structures of higher cost than portal frames for routine applications. Nevertheless, their use is cost-effective in particular occasions, such as when large spans over 60m need to be achieved, heavy roof loads need to be accounted during design or when deflections are critical.

The truss chords are typically made out of hot-rolled universal beam or column sections or structural hollow sections. The internal members can be angle, beams or hollow sections, depending on design loads, configuration and fabrication costs (BCSA, 2006). Trusses typically form a deeper girder than single beams or plate girders as those used in portal frames. Services can be integrated within the depth of the truss. The stiffness of the trusses is very high and deflections can easily to be controlled. They are a good option when a flat roof is needed, while long spans can be achieved and high loads can be supported whether flat- or pitched-roofed.

Lattice trusses in single-storey industrial buildings are typically pitched or flat, with the latter comprising uniform depth. Trusses can be connected to columns either with simple beam-column connections or with connections providing bending moment capacity for portal frame action (see Figure 2.3). The lateral stability of a trussed-roof frame is typically provided by vertical bracing at the gables and longitudinal horizontal bracing at the roof. Alternatively, when portal trusses are used, with the lateral stability is given by the connection between truss and columns which provides global bending moment resistance. The longitudinal stability is provided by transverse horizontal bracing at roof and vertical bracing which can be positioned either the gable bays or within the length of the structure. Three-dimensional trusses are another alternative.

Trusses can be designed according to BS EN 1993-1-1:2005 and the relevant UK National Annex or BS 5950-1:2000.

2.1.2.3 Northlight construction

Northlight construction is a variation of trussed roof construction and has been predominantly used in the past for large industrial-type buildings where daylight was essential. Northlights are typically constructed as a series of mono-pitch roofs in relatively short spans, comprising glazing units installed within the depth of the steeper (or vertical) truss slope. The system allows maximising natural light and is oriented north or north-east to minimise solar gains. It is common to design a conventional long-span truss on the steeper slope and running perpendicular to the plane of the north light truss.

An illustration of the north light truss arrangement is shown in Figure 2.4.

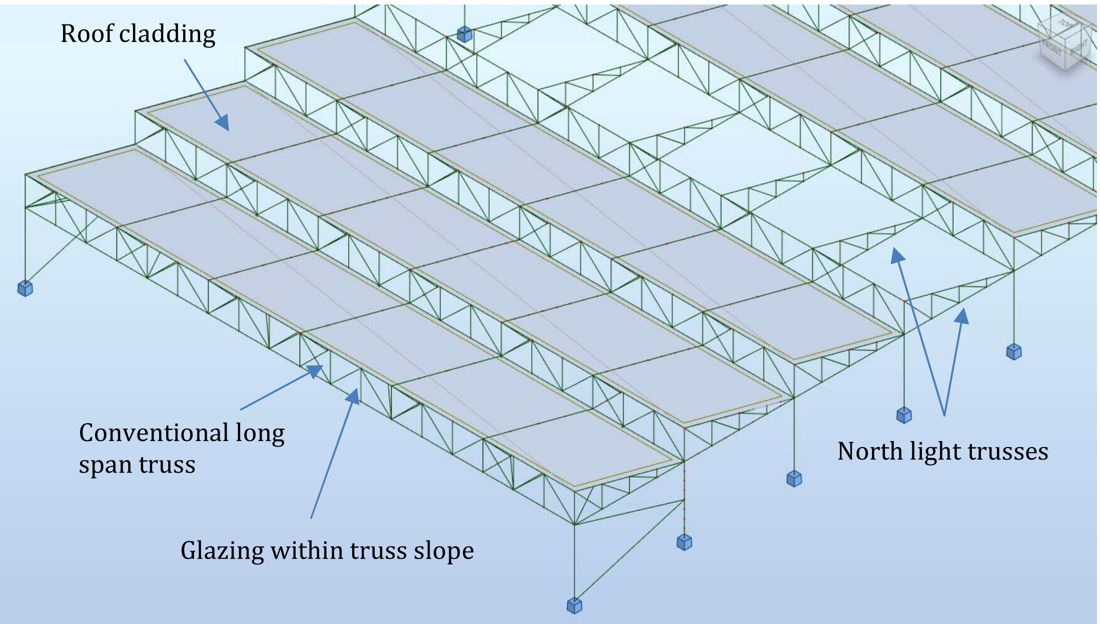


Figure 2.4 Northlight truss systems

North light trusses are designed to maximise natural lighting; however, the operational energy performance of the building would be fundamentally different compared to the typical pitched roof with roof lights. This is primarily in terms of increased risk for overheating and increase of internal building volume. The in-use energy performance of buildings with north lights is very critical for the design of the building and this will be discussed later in Section 2.2.4.3.

2.1.3 Secondary structure

Purlins and side rails are used as the secondary structure for the roof and walls respectively. The members are typically light gauge cold-formed components, with Zed, Sigma or Cee sections up to approximately 300mm deep and with gauge varying between 1.2mm to 3.2mm. Timber purlins are also possible but almost exclusively used for agricultural buildings.

Purlins and side rails span directly between rafters and columns respectively. They can be either single span or continuous, with various connection possibilities. The secondary steelwork supports the cladding and transfers the externally applied loads back to the primary frame, while it also transfers horizontal loads into the bracing systems. Moreover, the outer flange of the purlins and side rails is stabilised by the cladding, allowing the members to develop high bending moment capacities under compression, such as under gravity load. Anti-sag rods are used to stabilise the purlins and side rails under wind uplift and also during construction. The purlins themselves are also used to stabilise portal frames out-of-plane temporarily during erection.

For portal frames in particular, purlins and rafters have the critical role of providing restraint to the rafters and columns. The secondary steelwork restrains the outer flange of the rafters and columns at the point of fixings. Purlins are typically spaced at distances between 1.5m to 2.4m, with a spacing distance of 1.8m to 2.0m being the typical one in the majority of cases. Historically, this is for two reasons:

- Profiled metal cladding is designed to span in the direction from ridge to eaves in duo- or multi-pitch roofs with the orientation of the profiles allowing the rainwater to run-off. The typical profile depth to allow the water flow is 32mm, which corresponds to a max span of 2.4m.
- For plastic design, the maximum restraint spacing to the primary frame members is 1.8m based on the relevant formulas in the structural Eurocode (BE EN 1993-1-1:2005) or British Standards (BS 5950-1:2000). This is to allow the hot-rolled steel members to develop their full plastic capacity and develop plastic hinges. This is a requirement typically across the whole length of the rafter, since outer flange of the member can be in compression over a large length under the gravity loads. For elastic design, the members do not

necessarily require such onerous restraint requirements, since the heavier sections can be used to provide the required bending moment capacity.

Spacing and restraint requirements are less onerous for columns. Column restraints need to typically be provided mainly near the eaves and sometimes at column mid-height.

Moreover, purlins and side rails allow the attachment of 'stays' (often also referred to as 'knee-bracings') at discrete locations in order to provide restraint to the inner flange of rafters and columns as well (see Figure 2.5). This is very critical for the wind-uplift conditions, when the inner flange of the rafters or columns is in compression, hence restraints are required to reduce the buckling length and allow for development of the cross-sectional capacity for the member. Any solution to eliminate the secondary structure needs to meet these restraining requirements.

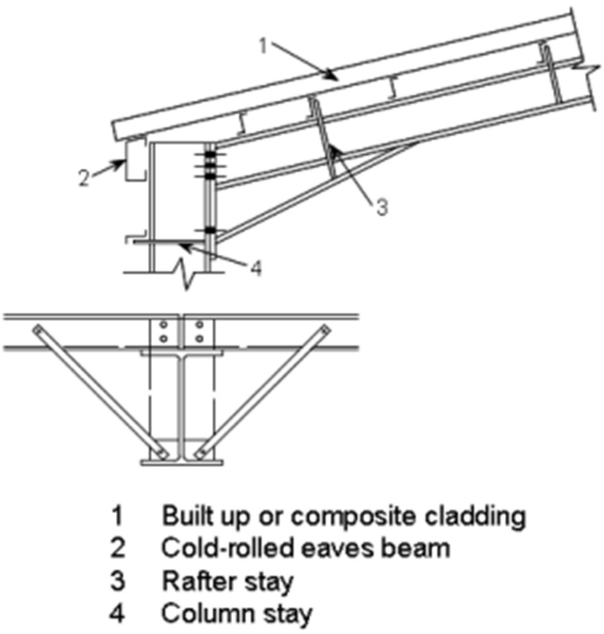


Figure 2.5 Column and rafter 'stays'; Source: Heywood (2006), steelconstruction.info – Image courtesy of the Steel Construction Institute

Steel purlins and cladding rails can be designed according to BS EN 1993-1-3:2006, BS EN 1993-1-5:2006 and the relevant UK National Annexes. In the UK, it is very common that approved calculations and test data are incorporated into load-span tables issued by manufacturers and made readily available to designers and specifiers.

2.1.4 Building envelope options

Industrial buildings commonly comprise profiled metal cladding for the building envelope (Heywood, 2006). Such systems are typically 'twin-skin' systems, comprising an inner and outer layer of metal sheeting with insulation in between. 'Single-skin' are far less common nowadays and are only be used where no thermal requirements exist. There are two major twin-skin building envelope systems:

- Built-up systems
- Sandwich (composite insulated) panels

Both systems demonstrate excellent thermal performance and they are designed to minimise thermal losses and thermal bridging. Standing seam roofs are also used, being essentially a different form of built-up system. Other systems such as structural decking for roofs or structural liner trays for walls are also available as long-span solutions; however, these are very rarely used in the UK any more. This is due to increased thermal bridging issues (for structural liner trays) and difficulties in attachment of 'stays' which are necessary to provide restraints to the inner flanges of portal frame members.

Both built-up and sandwich panel envelope systems are primarily used in conjunction with purlins and cladding rails. The current state of the art requires the cladding to restrain the outer flange of the purlins and cladding rails, which in turn provide restraints to the portal frame. Built-up and sandwich panel systems can provide the required restraint easily and they overall demonstrate excellent collaboration with the secondary structural elements. Since portal frames are the predominant form of construction and require the purlins and the cladding rails (and attached 'stays') to achieve the high structural efficiency, built-up and sandwich panel systems are also the predominant forms for the building envelope of single-storey industrial buildings.

A description of built-up and sandwich panel systems is provided herein. Detailed information is available by Heywood (2006).

2.1.4.1 Built-up systems

An illustration of a typical built-up cladding system is shown in Figure 2.6.

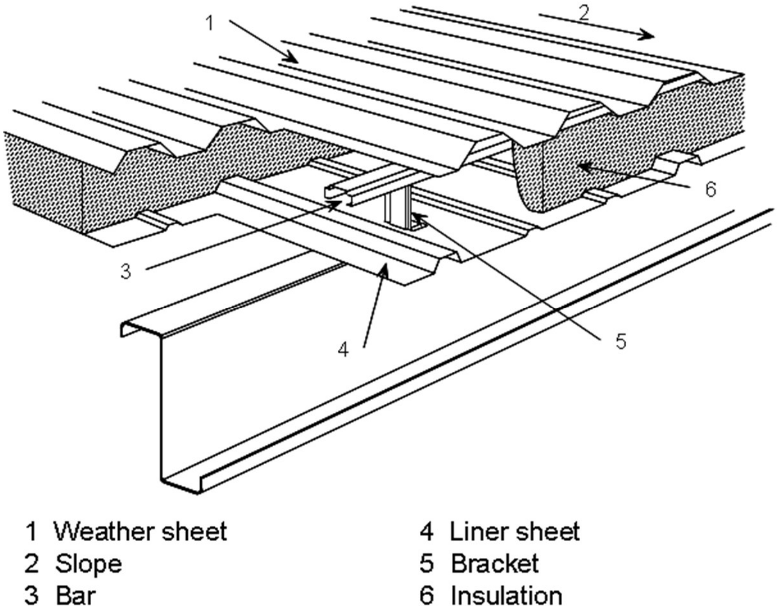


Figure 2.6 Built-up system; Source: Heywood (2006), steelconstruction.info – Image courtesy of the Steel Construction Institute

Built-up systems comprise an outer profiled sheet, a profiled inner (liner) sheet, insulation blocks in between and a light gauge steel spacer bar and a bracket system or halters for standing seam systems. The cladding system is built-up of these components on site and can be used for both roof and wall applications. The system is designed to span directly between purlins and cladding rails at short distances between 1.5m to 2.4m.

Built-up systems span from ridge to eaves between closely-spaced purlins or side rails, typically spaced at 1.8m for the reasons explained in Section 2.1.3. The orientation of the cladding profiles is such that it allows for the span distances to be easily achieved and for the rainwater to run-off. The system is applicable for pitched roofs to prevent water leaking through the joints of the outer sheet. The use of built-up systems together with secondary steelwork makes them ideal for portal frames where cladding rails and purlins (and attached ‘stays’) are required to provide the onerous restraint requirements to the primary members.

The standing seam system is another form of built-up system, used for roof applications, utilising an outer layer of steel or aluminium sheeting which is profiled to include a clipped joint between adjacent sheets. The system benefits from the elimination of the requirements for through-fixings and, consequently, can be utilised at low-pitch roof slopes (down to 1^{o}).

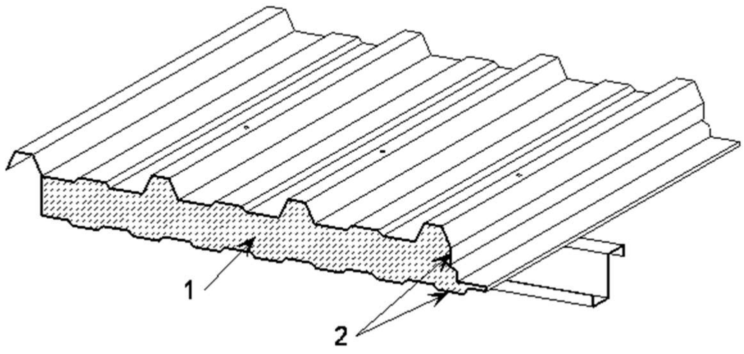
The liner sheet is fixed to the purlins or side rails and provides an airtight layer, the base for the thermal insulation and the restraint to purlins. The spacer bar supports the outer sheet at the required spacing, which is the span between purlins for roofs and cladding rails for walls. The steel brackets are used to provide support to the bar system and are firmly fixed to the purlins or side rails through the liner. Thermal pads are often used to minimise thermal bridging. The outer sheet provides a weather-tight layer and transfers the externally applied load (wind, snow, imposed) back to the purlins through the bar and bracket system.

The insulation layer is non-structural and is placed directly on the liner sheet on site. Mineral wool is the most common type, due to its low weight, low thermal conductivity, low cost and ease of handling. The typical insulation height has increased significantly during the recent years in order to address the demand for lower U-values s reflected in the recent revisions of Part L. For a U-value of 0.20W/m^{2}K, 210mm mineral wool insulation is required, while for a U-value of 0.15W/m^{2}K, the mineral wool insulation increases to 300mm.

The increase of insulation demand has increased the weight of the cladding assembly considerably during the recent years. Similarly, while brackets with increased heights are required to accommodate the increased insulation depths, heavier brackets with stiffer and thicker cross-sections for stability are used. This has added even more weight to the system and consequently to the supporting structure, while leading to more onerous thermal bridging issues. The insulation in built-up systems does not serve any structural purpose hence increased depths do not present any opportunity for further structural exploitation. For this reason, built-up cladding systems are not considered in this research.

2.1.4.2 Sandwich (composite insulated) panels systems

An illustration of a sandwich panel system is shown in Figure 2.7.



1 Insulation
2 Metal sheets

Figure 2.7 Sandwich (composite insulated) panel system; Source: Heywood (2006), steelconstruction.info – Image courtesy of the Steel Construction Institute

Sandwich panels represent a form of lightweight composite construction, comprising a rigid layer of insulation sandwiched between and adhered to two thin layers of metal sheeting, forming a single manufactured unit.

The metal faces are typically made of steel with a thickness range between 0.3mm and 0.7mm, or less commonly aluminium. Face geometry can be flat, micro-ribbed (often referred to as 'satinlined'), lightly profiled or fully profiled. Roof applications typically comprise a fully profiled external sheet and a lightly profiled internal (liner) sheet, while wall applications mostly comprise flat, lightly profiled or micro-ribbed geometries. The core is typically made of Polyisocyanurate (PIR), Polyurethane (PUR), Extruded Polysterene (EPS) or Mineral Wool (slabs or lamellas). In the UK, PIR and Mineral Wool are the most extensively used sandwich panel core materials, with the latter being used primarily for wall applications. PIR insulation is used both roof and wall systems. The insulation expands rapidly when sprayed on the metal layer and possess self-bonding properties which eliminate the need for adhesives. This increases the speed of production significantly. When mineral wool rigid insulation slabs are used, a layer of adhesive needs to be applied to provide the required bond between the core and the steel layers. PUR and EPS are used in the rest of Europe but not in the UK because of poor performance in fire.

The behaviour of a sandwich panel relies on the composite action between the insulation core and the bonded metal faces. One of the major functions of the core, besides its insulating ability, is the fact that it keeps the metal faces apart and increases their moment of inertia, enabling the assembly to act as a single beam. Furthermore, it increases the strength of the metal faces by providing a stabilising function against local failure whilst providing shear connection between them. As a general principle of the sandwich panel structural behaviour, the shear force is regarded as resisted by the core and the bending moment by axial stresses in the metal faces. The behaviour may, therefore, be regarded similar to an I-beam in which the metal faces correspond to the flanges and the core behaves as the web. As a result, the increase of the insulation depth in a sandwich panel leads to an increase of the component's stiffness, resistance and, consequently, bending moment capacity, in a similar manner to a beam with a deeper web.

Sandwich panels are currently specified to span between purlins or side rails, typically spaced at 1.8m for the reasons already explained in Section 2.1.3. When spanning between purlins, the direction of the sandwich panels is from ridge to eaves and the orientation of the profiles is such that it allows the rainwater to run off. The excellent combination of sandwich panels with secondary steelwork makes the system ideal for portal frames where onerous restraints need to be provided to the primary members. Sandwich panels can be through-fastened directly on the supporting structure. Seam fasteners for roof systems are typically required to provide weather tightness, while wall systems typically incorporate provisions for weather-tight joints without the need for seam fastening.

However, the increase of insulation thickness to minimise thermal losses as per the requirements reflected in the recent revisions of Part L, has led to engineering of modern sandwich panels systems with increased stiffness and load-carrying capacity. Modern systems possess spanning capabilities in excess of 6.0m for roofs and 8.0m for walls (discussed in more detail in Sections 2.4.3 and 2.4.4) for typical load magnitudes in the UK. Consequently, their capability is largely underutilised when components are required to span at 1.8m. A recent exception is for wall sandwich panels. Where the requirements are less onerous and restraint to the inner flange of columns is not needed, it is common that sandwich panel wall systems exploit their increased insulation depth and span directly between primary framing as long-span wall applications.

Consequently, the increased insulation depth and the mechanics of sandwich panel technology present an opportunity for further exploitation of the system's increases structural capability.

Sandwich panels for conventional applications can be designed according to BS EN 14509:2013 and the state-of-the-art manual by Davies *et. al.* (2001). In the UK. it is very common that approved calculations and test data are incorporated into load-span tables issued by manufacturers and made readily available to designers and specifiers.

2.1.5 Interdependence of envelope and structure and opportunities

As noted in Section 2.1.2.1, steel portal frames demonstrate high degree of structural and cost-efficiency and for this reason they have been the frame option of choice. The high structural efficiency of portal frames is due to the very strict restraint requirements provided to the rafters and columns by the purlins and side rails respectively. In turn, the structural efficiency of the purlins and side rails is due to the restraint provided by the envelope. The cladding is required to be capable of spanning between the purlins and side rails and adequate fixing needs to be provided. Since the purlins and cladding rails span directly between rafters and columns, the orientation of the cladding profiles allows strength and stiffness in the direction of span to be achieved by the cladding, while allowing the rainwater to run off. Overall, the interdependence of primary frame, secondary steelwork and building envelope is paramount to achieve structurally efficient portal frames.

As noted in Section 2.1.3, the onerous restraint requirements for plastically designed portal frames are met by spacing of the purlins at typically 1.8m, consequently requiring the roof cladding systems to span between at this distance. As mentioned in Section 2.1.4.2, highly-insulated modern sandwich panels to comply with Part L thermal requirements can span at distances far greater than those required due to purlin spacing. Hence, the current technology for portal frames significantly underutilises the structural capability of sandwich panel systems. While portal frames offer the highest structural efficiency for the frame, the current construction technology does not necessarily offer the highest structural efficiency for the frame and envelope together.

Consequently, there is an opportunity to reengineer the frame to exploit the increased structural capabilities offered by modern sandwich panel systems possessing increased insulation for thermal performance purposes. This would require an examination of the options which offer highest structural efficiency for the frame and building envelope together. The identified opportunities to be investigated further are shortlisted in Section 2.3.

Advancing the building envelope to replace functions of the primary structure would require consideration of the current procurement framework for single-storey industrial buildings in the UK. Currently, the structure and the building envelope are typically specified by different parties; the structural engineer or steelwork fabricator specifies the structure, while the cladding contractor specifies the building envelope. Using the envelope for advanced structural capabilities may require a closer coordination between the structure and cladding specifiers.

As highlighted in the introduction of the current section, any structural material savings in single-storey industrial buildings would be likely to have a significant impact in this market sector in terms of embodied carbon emissions. A review of carbon emissions-related aspects in the UK for non-domestic and single-storey industrial buildings, as well as the trends, role and opportunities related to the building envelope are discussed in Section 2.2.

2.2 Reduction of carbon emissions

As discussed in Chapter 1, the UK Climate Change Act (2008) has adopted a strict environmental agenda for the UK to reduce its operational carbon emissions by at least 80% by 2050 against the 1990 levels (The Stationary Office, 2008) and comply with the Kyoto Protocol (United Nations, 1998). Buildings in the UK are responsible for more than 50% of the operation carbon emissions (BRE, 2006). The main regulatory means towards achieving the EU energy efficiency requirements in the building sector is the Energy Performance of Buildings Directive (EPBD) (European Parliament and Council, 2002), which UK has adopted. The effort of the UK to reduce its operational carbon emissions in buildings is largely reflected within its national regulatory framework.

The UK Government has previously set the ambitious plan for new non-domestic buildings to be 'Zero Carbon' in operation by 2019 and domestic buildings by 2020. Although the UK government has now put these deadlines on hold on economic grounds (HM Treasury, 2015), there is still a requirement that the UK abides by its legally binding targets to reduce carbon emissions according to the UK Climate Change Act (2008) and Kyoto Protocol (United Nations, 1998). Given the dominating contribution of buildings in the UK to the country's operational carbon emissions, it is almost certain that achievement of 'Zero Carbon' buildings will inevitably be required to be achieved in the future in order to realise the country's legally binding targets operational carbon emissions reduction (The Stationary Office, 2008).

During the recent years, regulatory approaches for buildings have been adopted in the Part L of the Building Regulations (HM Government) to assist towards achieving the agreed carbon reduction targets for buildings on the trajectory towards 'Zero Carbon'.

The current section presents an overview of the UK regulatory context for carbon emissions, the trend of the future likely energy conservation requirements, the environmental performance of single-storey industrial buildings, the role of the building envelope and the related opportunities for embodied carbon emissions reduction.

2.2.1 Operational and embodied carbon emissions

The carbon emissions associated with the performance of buildings are categorised into 'operational' and 'embodied'. According to IPPC (2007), the main measures to reduce the greenhouse gas emissions of buildings are decreasing the operational and embodied carbon and employ renewable energy sources for the building's functions. Reducing the operational carbon emissions of buildings has been the main focus of sustainable building design and innovation in the UK and internationally, while the appreciation of the embodied carbon importance has been continuously increasing.

'Operational carbon' is the term describing the quantity of greenhouse gas emissions occurring during the operational phase of the buildings. These emissions are categorised into regulated and unregulated. Regulated emissions arise from energy consuming activities, such as heating, cooling, ventilation and lighting and for England and Wales are regulated under the Approved Document Part L of the Building Regulations. Unregulated emissions arise from activities such as appliances use (e.g. for cooking) and small power plug loads (e.g. IT equipment); designers generally neglect the unregulated energy emission quantities since control over their use is very limited.

'Embodied carbon' emissions are those occurring during the manufacturing and transportation of the construction materials and components, the construction process and the end-of-life aspects of the building (i.e. demolition, recycling and reuse of materials).

The embodied and operational carbon emissions together make up the complete lifecycle carbon footprint of the building.

The Approved Document Part L of the Building Regulations in the UK, as well as comparable international standards, provides performance standards solely for the reduction of operational energy and carbon in buildings. The contribution of embodied carbon is not currently regulated and neither taken into account in the qualification of the environmental performance of buildings within the current regulated context.

2.2.2 Approved Document Part L and carbon reduction targets

The Approved Document Part L – Conservation of Fuel and Power of the Building Regulations is issued by the UK Government and provides general and practical guidance to achieve compliance with the energy efficiency requirements set in the Building Regulations, in line with the carbon and energy reduction targets and legislation. The first Part L was issued in 1985 and since then it has been revised in 1991, 2002, 2006, 2010 and 2013. An addendum to the 2013 revision was also issued in 2016. A new revision is anticipated in 2019.

Part L consists of four parts: Part L1A: for new dwellings; Part L1B: for existing dwellings; Part L2A: for new buildings other than dwellings; and Part L2B: for existing buildings other than dwellings. Regulations for new industrial (i.e. non-domestic) buildings, which are the focus of this study, are covered by Part L2A.

The Approved Document Part L plays the key role of regulating operational carbon emission requirements for buildings in the UK while setting the intermediate carbon reduction targets on the trajectory towards 'Zero Carbon' buildings and achieving the state's carbon reduction commitments. Every revision of the document provides an operational carbon reduction target for the regulated emissions against the previous version. In 2006, Part L required a carbon reduction of 23.5% or 28% for non-domestic buildings. Since then, carbon reduction targets against Part L 2006 have been introduced. A summary is shown in Table 2.1. More strict requirements in terms of energy conservation are anticipated to be imposed in the next revisions (DCLG, 2011, 2012).

Table 2.1 Carbon reduction requirements between Part L revisions for non-domestic buildings; Source: HM Government, DCLG (2011, 2012)

Part L revision	Reduction of regulated carbon emissions to 2006 levels
2006	Base case
2010	25%
2013	44%
2016 (postponed)	70% (possible)
2019 / Zero Carbon deadline	100% (possible)

On the trajectory towards achieving the carbon reduction targets, the Part L revisions in 2006, 2010 and 2013 adopted an 'aggregate approach' in recognition of the variation of non-domestic building types, their corresponding energy demand and the associated cost effectiveness of solutions to achieve carbon reduction percentages. The aggregate approach sets an overall operational carbon reduction rate over a mix of various building types (e.g. offices, warehouses, schools etc.) taking into account projections of the building mix during the forecasted periods (DCLG, 2011, 2012). Based on this approach, warehouses (retail and distribution), together with office buildings, need to contribute much higher reduction rates than the average of the building mix (DCLG, 2011, 2012). Table 2.2 shows the contribution of retail and distribution warehouses in the aggregate operational carbon emissions of the building mix for different reduction targets to Part L 2006. Although the aggregate targets agreed in 2012 (DCLG, 2012) are now obsolete and new ones are expected to be set, the requirement for increased contribution of savings by the warehouse buildings is evident.

Table 2.2 Warehouse carbon reduction contribution over non-domestic building mix; Source: DCLG (2011)

Building type	**Operational carbon reduction to Part L 2006**		
	2013	**2016**	**2019**
Retail warehouse	44%	54%	60%
Distribution warehouse	55%	66%	72%
Aggregate (non-domestic building mix)	33%	41%	49%

Note: aggregate targets are now obsolete.

Hence, it may be highlighted that warehouse building types, which are the most frequent use of single-storey industrial buildings, have a major contribution towards reducing the aggregate operational carbon emissions for the non-domestic building mix.

2.2.3 Approved Document Part L compliance and the role of the building envelope

The earlier versions of the Building Regulations and Part L included provisions for reduction of energy consumption by introducing prescriptive requirements for the heat loss rates (U-values) of building components and the building envelope as a whole. Increasingly tighter performance requirements were introduced over the years, primarily demanding lower U-values of the building envelope. These requirements were primarily addressed by increasing the insulation thickness of the envelope and limiting the glazing area of the façade.

Since 2002, significant modifications have been implemented to Part L. These were made to reflect the recognition that significant amount of carbon emissions are due to the operational use of buildings and factors other than the U-values of the envelope alone are important to achieve energy-efficient buildings. Such factors include limits for air-tightness, integration of the building envelope with heating, cooling and air-conditioning requirements, as-built inspections and monitoring processes. Nevertheless, the 2002 revision still required increased thermal insulation standards by using improved U-values.

Since 2006, Part L moved away from prescriptive element values and adopted a holistic approach in terms of the energy and carbon performance of the building. Compliance can be achieved through the overall performance of the finished building, taking into account several factors, such as the building's design, the envelope performance, the efficiency of the building services and, finally, the utilisation of low carbon energy sources. The designer can achieve the required performance by taking into account the implication of each of the above-mentioned factors within the whole building performance and without being limited by a prescriptive route. Despite the various factors which need to be considered, the energy efficiency of the building envelope is still paramount and this is reflected by the fact that U-values continued to reduce.

Overall, while there is recognition that many factors influence energy efficiency of a building, minimising thermal losses through the building envelope remains of primary importance. This has been recognised by Part L in each revision by prescribing lower U-values. While the Part L revisions since 2006 do not prescribe U-values, minimum backstop values for critical elements have been introduced together with recommended values for a notional building. The U-values for the notional building are provided as guidance and their use is likely to provide the minimum performance requirement for a typical building, however, without taking into account the building-specifics. It is ultimately up to the designer to specify the combination of operational carbon reduction measures and the optimum U-values and insulation thicknesses for the building envelope.

A summary of the evolution of the U-values in Part L for non-domestic buildings and the current status is shown in Table 2.3. This shows that more demanding energy efficiency standards for the building envelope are set in every new revision, while the improvements compared to the early Part L versions have been significant. While there is flexibility over the combination of measures adopted to provide the required building performance, it is evident that the demand for the building envelope in terms of U-value has been increasing. Consequently, with the available building envelope technology, there is a need for increased insulation levels in the building envelope compared to the past.

Table 2.3 Evolution of Backstop and Notional Building U-values for planar elements in Approved Document Part L of the Building Regulations; Source: HM Government

Element	1985	1990	1995	2002	2006		2010		2013/2016	
					Notional Building	Backstop value	Notional Building	Backstop value	Notional Building	Backstop value
Wall (W/m^2K)	0.70	0.45	0.45	0.35	0.35	0.70	0.26	0.35	0.26	0.35
Roof (W/m^2K)	0.70	0.45	0.45	0.25	0.25	0.35	0.18	0.25	0.18	0.25
Rooflights (W/m^2K)	5.7	5.7	3.3	2.2	2.2	3.3	1.8	2.2	1.8	2.2

DCLG has been the main advisory body for setting the Part L requirements. DCLG (2013) examined the optimal energy efficiency measures based on operational carbon emissions and capital costs for a mix of non-domestic buildings on the trajectory towards 'Zero Carbon' non-domestic buildings. For a retail or distribution warehouse it was shown that the cost-optimal combination of energy efficiency measures includes the following improvements in the heat-loss rates of the building envelope:

- Roof U-value: $0.15W/m^2K$
- Wall U-value: $0.25W/m^2K$

This denotes that building envelopes with these tighter U-values would be likely to be required in the future in order the operational carbon reduction targets to be met. With the available building envelope and insulation technology, the requirement for lower U-values is addressed by increasing the insulation. Table 2.4 shows the depths required to achieve various U-values for the two most commonly used insulating materials, PUR/PIR and Mineral Wool. For example, achieving the 2013 notional building U-value for roof of $0.18W/m^2K$ requires 140mm of PIR or 200mm of mineral wool.

Table 2.4 U-values and typical insulation thicknesses

Insulation	U-values (W/m^2K)						
	0.35	0.26	0.25	0.20	0.18	0.15	0.10
	Insulation thickness (mm)						
PUR/PIR	70	90	95	120	135	155	240
Mineral Wool	100	135	140	180	200	240	360

2.2.4 Carbon emissions of industrial buildings

Recent studies on the environmental performance of modern non-domestic buildings in the UK were undertaken by Target Zero (2011a, 2011b). The studies aimed to provide guidance on the design and construction of sustainable, low and zero carbon non-domestic buildings in the UK and included assessments and comparative studies in terms of sustainability performance of various types of recently constructed non-domestic buildings. The real buildings were slightly modified to the minimum Approved Document Part L 2006 requirements to provide a consistent benchmark and were used as case studies. The studies included investigation of the buildings' environmental performance in terms of operational carbon and embodied carbon, together with other sustainability and cost aspects.

Two large single-storey industrial building types were included in the assessment: a distribution warehouse and a supermarket building. The structure for the base case buildings were a steel portal frame for the warehouse building and a braced steel frame for the supermarket building. Alterative structural options including concrete structure with glulam beams and steel frame with northlight construction were also examined. With the exception of the northlight construction, the other two options comprised rooflights.

2.2.4.1 Operational carbon emissions

Target Zero (2011a) showed that for a typical warehouse building complying with Part L 2006 requirements, the most significant contribution in the total operational carbon emissions is associated with lighting at 73%. Heating was also significant at 11%. The examined warehouse was predominantly heated, hence emissions associated with cooling energy were not estimated. Target Zero (2011b) also examined a supermarket building, with its operational performance requirements being largely different, including cooling requirements, longer hours of operation and higher demand for lighting. The study showed that lighting also had the greatest contribution at 49%, nevertheless heating and cooling being also significant at 7% and 8% respectively. Notably, control of heating and cooling are largely associated with the thermal performance of the envelope and reduction of thermal leakage.

The study also showed that the influence of the structural option on the operational carbon emissions is minimal (3%). This variation was primarily related to the different depths of the alternative structural forms and the consequent differences in the internal volume. For example, when the internal volume was increased, the requirements for heating increased as well, while those for cooling decreased. Furthermore, the northlights roof option showed that the risk of overheating can be significantly reduced.

For the warehouse building case study, Target Zero (2011a) estimated that the most cost-effective energy efficiency measures to achieve ‘Zero Carbon’ status would include improvements to the building envelope’s heat-loss rates and proposed the following:

- Roof U-value: 0.10W/m^{2}K
- Wall U-value: 0.15W/m^{2}K
- Rooflights U-value: 0.90/m^{2}K

These estimations support the Part L changes and indicate that further improvements in insulation levels are likely to be required in order to achieve cost-optimal ‘Zero Carbon’ buildings.

A similar exercise was undertaken for the supermarket building, however the building envelope was found to be of less importance among the cost-efficient energy efficiency measures (Target Zero, 2011b). This is because the carbon emissions were primarily associated with improvements in lighting, hence improvements in the building envelope were of less importance. The study found that improved U-values for the wall envelope 0.25W/m^{2}K would be required as part of the cost efficient energy measures to achieve Zero Carbon; however no more envelope provisions were included in the measures package.

2.2.4.2 Embodied carbon

Target Zero (2011a, 2011b) estimated the embodied carbon emissions for the selected warehouse and supermarket buildings case studies. The adopted methodology used a cradle-to-grave approach, taking into account the end-of-life impacts. The following stages were included: material and product manufacturing; transportation; construction waste; transport of waste from site; construction process; maintenance; demolition or deconstruction; and end-of-life recovery rates. The building elements included in the assessment were: foundations, ground floor slab and infill materials; superstructure; external and internal walls; roof; windows and roof lights; drainage; and external works.

For the warehouse base case scenario (portal frame structure), representing the current best practice and the option with the lowest total embodied carbon (Target Zero, 2011a), the study showed that the concrete floor slab and foundations where dominant in terms of embodied carbon contribution (45%). The carbon emission rate for the concrete is significantly less compared to steel; however the large volume used makes a significant contribution to whole construction project. Despite its large contribution, the floor slab has little scope for further improvements. The steel frame contribution was 12% and the roof and wall cladding 16% (notably with higher contribution than the bearing structure). Consequently, the structural frame and cladding together were found to yield the highest embodied carbon (28%) after the concrete foundations and floor slab. Although the concrete used for slabs and foundations was not shown separately, it may be highlighted that the size of the foundations is highly dependent on the weight of the structure and envelope; a lighter superstructure would lead to lower embodied carbon in the foundations. In addition, site works for the development (such as hard-standings or landscaping) were found to have a large embodied carbon contribution (21%), although not strictly being part of the building. On the other hand, drainage and onsite activities were found to have a negligible impact of 3% and 1% respectively. The results for the supermarket building were similar (Target Zero, 2011b).

The embodied carbon assessment is highly dependent on the assumptions. Earlier studies by Sansom (2007) on a large warehouse in the UK showed that the steel frame contribution in the embodied carbon emissions may vary between 12% to 23%, depending on whether end of life aspects (recycling, reuse) are taken into account. When end of life aspects are included, the embodied carbon impact of the frame was lower and the comparative contribution of the various element types was generally very similar to Target Zero (2011a).

Target Zero (2011a, 2011b) showed that the cladding, bearing structure and foundations together have a major contribution in the embodied carbon of the building and they are critical elements that should be taken into account if low embodied carbon construction is pursued in the future. Despite the holistic embodied carbon review, Target Zero (2011a, 2011b) merely compared different structural arrangements but did not examine opportunities for reducing the embodied carbon.

2.2.4.3 Operational energy performance for northlights construction

The energy performance of buildings with northlights construction is fundamentally different compared to traditional single storey building construction with rooflights. Northlight construction has a series of advantages, which are discussed below and will be qualitatively considered in Chapter 8. These are:

- They diffuse light to enter the middle of the building and they reduce the amount of direct solar radiation, without the requirement of solar control. Consequently, they improve the consistency and uniformity of light without the risk of overheating (Target Zero, 2011a, 2011b). Light-reflective interior surfaces may also enhance consistency (Kendrick and Wang, 2012, Kendrick *et. al.*, 2012).

- Their orientation is such that high solar gains are avoided. This is ideal when low temperatures and / or mechanical cooling is required.
- The south facing slope of the roof creates ideal surfaces to install photovoltaic (PV) panels. The angle orientation of the PVs may also increase their output, when compared to low slope orientations. According to Target Zero (2011a, 2011b), the optimum pitch of PVs in countries like the UK is between 30^{o}-35^{o}, which may increase their annual output by 10%. Moreover, since the northlights are placed within the depth of the truss, they leave the full roof surface available for PV installation, whereas for roofs with rooflights the available area is reduced.
- They are ideal for natural ventilation when openings are installed due to their high positioning and assistance from wind-pressure (Kendrick and Wang, 2012, Kendrick *et. al.*, 2012).
- Due to their vertical installation they allow water run-off and they are self-cleaning, whereas horizontally-positioned rooflights require more cleaning maintenance (Kendrick and Wang, 2012, Kendrick *et. al.*, 2012).
- Northlights require more heating energy per annum compared to rooflights, due to increased glazing area and decrease in beneficial solar gain during cold seasons. However, there is much less demand for cooling energy during warm months due to practically no risk of overheating. The study by Kendrick and Wang (2014), Kendrick *et. al.* (2012) showed that for a medium-sized building (2,904m^2), annual heating demand increases by 9%-12%, while for cooling it decreases by 70%. Target Zero (2011a) showed the same trend for a case study building, but with different magnitudes (35% increase for heating and 29% decrease for cooling).

There is an additional strong argument for the use of northlights, relating to climate change. It is anticipated that due to temperature rise, by 2050 the annual requirements for heating will decrease by 26% and for cooling will increase by 32%-38% (Target Zero, 2011a). This means that heating will become less important and cooling will become more, compared to the current requirements. If this projection is realised, then a good case for energy savings will be through the use of northlights. However, it is worth noting that if there are lower heating requirements during the winter then renewable technologies will be less beneficial in supplying heat (Target Zero, 2011a).

In terms of lighting, large northlight areas will generally increase natural light. However, there will be point where lighting benefits will be negated by the increased thermal losses through the glazing area (since U-values of glazing are higher than of the cladding) and, consequently, more heating will be required. Furthermore, the study by Kendrick and Wang (2014), and Kendrick *et. al.* (2012) showed that for equivalent percentage of glazing areas, the northlights would admit less light than rooflights. Nevertheless, increased glazing height for northlights can easily yield the high daylight factors required for light-demanding environments. These can be further strengthened by glazing of high light transmission, as well as through the use of light reflecting internal surfaces. It is worth highlighting at this point that energy for lighting vastly dominates the total operation carbon emissions according to Target Zero (2011a, 2011b) (73% for warehouse and 49% for supermarket case study).

Finally, northlight construction is favoured where high site temperatures (for reduced overheating, excellent cooling effects) and clear skies (for increased daylight use) are present. Their use may, therefore, be ideal for southern climates.

Overall, the use of northlights is ideal to reduce cooling, solar gains and avoid overheating in the building, while it is has lighting benefits (uniformity, consistency and intensity for large glazing areas) compared to rooflights. Where cooling is not required, rooflights are probably better in terms of beneficial solar gains; however, there is a high risk of overheating.

2.2.4.4 Embodied versus operational carbon

The regulatory requirements in the UK, as well as comparable international standards, have so far been focusing on the reduction of operational carbon emissions from buildings, with the relevant performance standards set primarily in Part L of the Building Regulations. However, while buildings require less energy in-use, the significance of the embodied energy of materials has increased. The increase of buildings' energy conservation efficiency in terms of heating and cooling is largely achieved by the increase of insulation in the building envelope to achieve lower U-values. In addition, employment of renewable energy for the reduction of operational carbon has been introducing additional material in buildings. Overall, in the effort to reduce operational carbon emissions, certain energy efficiency measures require higher usage of materials and, consequently, embodied carbon.

Sansom and Pope (2012) and Target Zero (2011a, 2011b) showed the ratio between the annual operational carbon emissions for different reduction targets and the embodied carbon. An extract of those results is presented in Table 2.5. For example, for a distribution warehouse constructed to Part L 2013 standards, the embodied carbon emissions are exceeded after 12.5 years of operation. This period would increase to 53.8 years if 100% of the regulated carbon emissions are reduced.

Table 2.5 Operational – embodied carbon ratio for warehouse and supermarket buildings; Source: Sansom and Pope (2012), Target Zero (2011a, 2011b)

Part L revision	**Operational carbon reduction target to Part L 2006 (in regulated carbon emissions)**	**Annual operational carbon : embodied carbon (years)**	
		Distribution warehouse	**Supermarket**
Part L 2006 compliance	Base case	7.8	5.0
Part L 2010 compliance	25% reduction	9.9	6.3
Part L 2013 compliance	44% reduction	12.5	7.7
N/A	70% reduction	19.4	11.3
N/A	100% reduction	53.8	24.0

The results by Sansom and Pope (2012) and Target Zero (2011a, 2011b) are specific for the analysed case studies and indicative; nevertheless, they clearly illustrate that as the operational carbon efficiency increases, so does the relative importance of embodied carbon. Consequently, on the trajectory towards low carbon or 'Zero Carbon' buildings, the embodied carbon would also need to eventually reduce.

More attention has been paid recently to quantifying and reducing the embodied carbon of buildings and construction products and there are strong arguments that total carbon emissions in terms of combined operational and embodied during the lifecycle of the building should be taken into account in the future. The Green Building Council in Australia (GBCA) (2008) addressed that while the construction industry is focusing on offsetting the emissions through operations to produce 'Zero Carbon' buildings, the main challenge is to produce 'truly carbon neutral' buildings, accounting for the environmental costs of both construction process and operational performance (Renewables the Key to Carbon Neutral Building, 2009). Nassen *et. al.* (2007) considered that carbon neutrality is achieved through the effect of two main stages of the building's life: the building construction and the occupancy. Boake (2008) concluded that 'carbon neutral' buildings should cover carbon emissions from energy consumption for materials and construction process, operating energy use, and those from the tenant activities associated with the building such as transportation requirements'. According to Chen *et. al.* (2011), the building's whole life assessment needs to comprise nine stages: building construction, fitment, outdoor facility construction, transportation, operation, waste treatment, property management, demolition and disposal for buildings.

Gustavsson *et. al.* recommended that all these stages are required to be included when evaluating the whole-life environmental impacts of the building and highlighted that a minimisation of the consequent carbon emissions should be pursued.

2.2.5 Combined operational and embodied carbon studies

The aforementioned earlier studies included considerations of the increasing importance of the embodied carbon emissions arising from use of materials to minimise operational carbon emissions. However, they did not include considerations of the interdependence between operational and embodied carbon efficiency. Where low U-values are required, the volume of insulation material is such that its embodied energy can begin to outweigh the saving in operational energy. Resalati (2015) examined for first time the points where the minimum total carbon (operational and embodied) occurs, above which the increase in embodied carbon of the thermal insulation outweighs the operational carbon savings. The context of the study was on residential, office and industrial buildings. Figure 2.8 shows the operational, embodied and total carbon emissions for a typical warehouse building with envelopes using polyurethane (PUR) insulation and over a given life span. The graph illustrates that there is a thickness for which minimum total carbon occurs, above which the increase in embodied carbon outweighs the operational carbon savings.

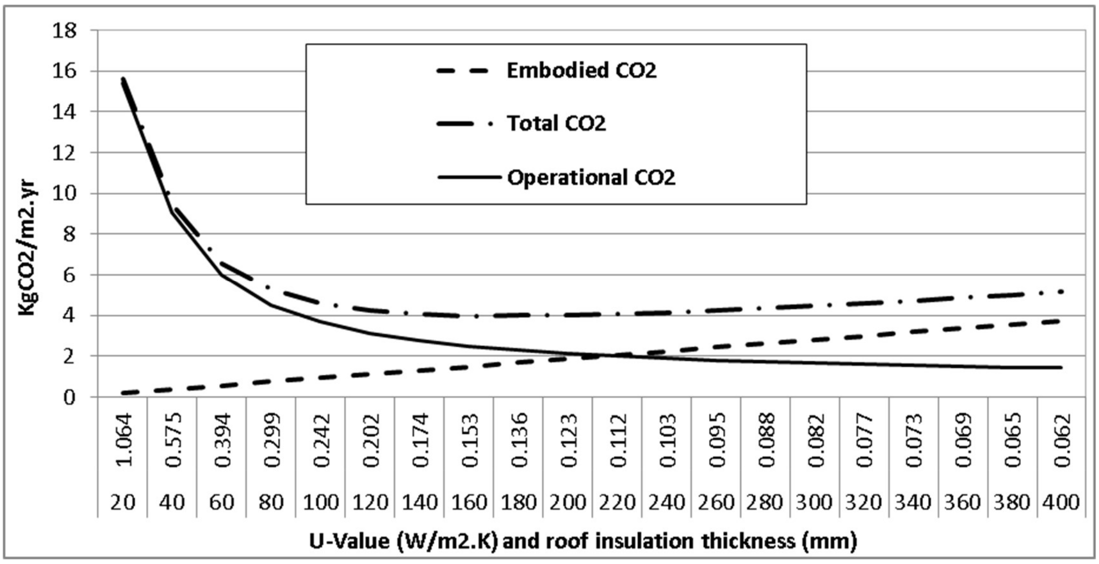


Figure 2.8 Typical embodied, operational and total carbon emissions for warehouse building with PUR insulation; Based on: Resalati (2015); Image source: Moutaftsis *et. al.* (2015b)

Resalati (2015) highlighted that an optimum insulation depth exists, beyond which any further increase in insulation would be illogical and counter-productive, while higher embodied energy insulation levels become more difficult to justify as they more rapidly lead to a carbon disbenefit. According to Resalati (2015), there are strong arguments that *'in the future a greatest parity between operational and embodied carbon will be achieved'*. The study suggested that a sensible maximum level of insulation should be incorporated into national building regulations and standards and sets limits to the amount by which current approaches to energy thrift can be escalated.

For a series of case studies, including warehouse buildings of various sizes, Resalati (2015) investigated the levels of insulation to achieve minimum total carbon over various building life spans. In addition, the research examined the cost-optimal building envelope insulation thickness when considered alone and in combination with photovoltaic panels. These are summarised in Table 2.6.

Table 2.6 Cost- and carbon- optimal insulation levels for warehouse buildings according to Resalati (2015)

Building type	**Insulation material**	**Proposed U-values**		
		Cost-optimal (building envelope alone)	**Cost-optimal (building envelope + PV)**	**Carbon-optimal (building envelope + PV)**
Distribution warehouse	Mineral wool	Roof: 0.13-0.20W/m^2K Wall: 0.18-0.27W/m^2K	Roof: 0.15W/m^2K Wall: 0.21W/m^2K	Roof: 0.07 W/m^2K Wall: 0.10 W/m^2K
	PUR	Roof: 0.18-0.27W/m^2K Wall: 0.14-0.24W/m^2K		Roof: 0.11 W/m^2K Wall 0.15 W/m^2K
Retail warehouse	Mineral wool	Further improvements not justifiable.		Roof: 0.10 W/m^2K Wall 0.14 W/m^2K
	PUR		Further improvements not justifiable.	Further improvements not justifiable.

Similar to the Target Zero (2011a, 2011b) findings, Resalati (2015) also concluded that for a retail warehouse where the energy consumption is primarily related to lighting, improvements in the thermal performance of the building envelope would be hardly justifiable.

Although Resalati (2015) introduced limits in insulation thickness increases considering total carbon approaches and minimum aggregates emissions, also showed that further improvements into the thermal performance of the building envelope would still be likely in order to minimise total carbon, if combined carbon approaches are eventually adopted. Despite adopting a different approach, this is also in line with Part L in terms of suggesting that improvements in the thermal performance of the building envelope would be required to achieve carbon efficiency. This is particularly true for distribution warehouses which are predominantly heated and the contribution of the thermal performance of the building envelope in the building's energy efficiency is considerable.

The earlier studies by Resalati (2015) pioneered in holistically examining the impacts of insulation level increases on the operational carbon emissions of typical buildings and the increased embodied carbon in the envelope and identifying limits where additional insulation can be counter-productive. However, neither Resalati's (2015) nor any of the aforementioned studies examined:

- the impacts of increased insulation thickness / adoption of renewables on the structure and
- the structural benefits and opportunities of increasing the insulation.

2.2.6 Impact of building envelope on structure

The impacts of increasing the insulation and installing photovoltaic panels to enhance operational energy efficiency on the structural frame weight of single-storey industrial buildings were earlier examined at the Steel Construction Institute (SCI).

The required reduction of U-values in line with the Part L requirements have been introducing higher insulation values to the building envelope of industrial buildings. These lead to onerous consequences in terms of increasing the envelope weight, requiring stiffer and heavier brackets in built-up systems for stability. Yandzio and Heywood (2012) examined the effects of increasing the insulation depth on the stability of brackets of built-up systems and estimated that increased bracket gauges or re-engineered cross-sections are required to accommodate current and future U-value requirements. These have been increasing the component's weight and consequently adding more weight to built-up envelope systems, apart from the weight of the insulation.

Furthermore, as the insulation thickness and cladding weight increase, the frame is required to resist higher dead loads to the point that design may require switch to heavier section sizes. Cooper-Smith and Heywood (2009) and Moutaftsis and Heywood (2012a) at the SCI examined the impact of proposed increases in insulation thickness of built-up systems on the member sizes of plastically designed portal frames and steelwork weight.

Cooper-Smith and Heywood (2009) examined the impact of reducing roof U-values from 0.25W/m^{2}K to 0.10W/m^{2}K. The study was focused on three small to medium size buildings with relatively high eaves heights. The study showed that frame weight increases of up to 7% for the medium-sized buildings are possible by moving from U-values of 0.25W/m^{2}K to 0.15W/m^{2}K as a result of the need to select heavier rafter and column sections. No further increases were found when using U-value of 0.10W/m^{2}K.

Similarly, Moutaftsis and Heywood (2012a) also examined the increase of the total envelope mass (including use of stiffer and heavier bracket systems) and purlins weight due to demand for lower U-values from 0.25W/m^{2}K to 0.10W/m^{2}K and the impact on three single storey industrial buildings (small, medium and large) with plastically designed portal frames. The frame heights were lower compared to those adopted in the study by Cooper-Smith and Heywood (2009); hence stability effects which may require changes in section sizes were less critical. The analysis showed that the increase of the cladding mass due to insulation thickness alone would not lead to meaningful increase of frame weight. However, a demand for a low U-value of 0.10W/m^{2}K in combination with the use of a heavier liner (for strength and stiffness purposes) may lead to an increase of the steelwork weight in the medium and large size building, up to approximately 8.5%. The steelwork weight increase was very small for a combination of U-value of 0.15W/m^{2}K and heavier liner.

Moutaftsis and Heywood (2012b) also examined the impact of the installation of photovoltaic panels on the frame and steelwork weight of three typical single-storey industrial buildings (small, medium, large). Three areas of concern were investigated: increased dead load on roof, asymmetric loading due to installation on one roof slope and increase wind loading. The study considered roof-integrated and in-plane roof-mounted solar panel types, installed on either one or both roof slopes in order to account for possible asymmetric loading effects. The study concluded that roof-integrated and lightweight roof-mounted PV systems had little or no impact on the design of the primary frame. However, the addition of medium or heavy weight roof-mounted PVs resulted in an increased frame weight of up to 8%. Installation of PV panels on both roof slopes was more onerous than installation on one slope only, due to the higher total load on the roof, while the asymmetry of loading one slope only did not appear to have an adverse impact on the frame.

Although all the aforementioned studies showed a strong appreciation of the impacts of increasing the material in the building envelope on the structure and the frame weight, they did not examine the opportunities arising from increasing the insulation and the potential structural benefits for the envelope and structure as a whole.

2.3 Opportunities

As discussed in Section 2.2, recent changes to Part L of the Building Regulations have been demanding lower U-values to minimise energy losses and operational carbon emissions in buildings. This has been leading to increased insulation thickness in the building envelope and consequently higher material usage and embodied carbon in the building. Despite strong evidence of diminishing marginal returns in terms of operational energy savings coupled with increases in the embodied carbon of the insulated material this trend looks set to continue.

As discussed in Section 2.1, the current state-of-the-art for single-storey industrial buildings requires building envelope systems to span short distances, typically 1.8m, to allow for provisions of onerous restraints to the highly structurally optimised portal frames. However, as mentioned in Section 2.1.4.2, highly-insulated modern sandwich panels to comply with Part L thermal requirements possess structural capability in terms of strength and stiffness which allows them to span distances far greater than those required due to purlin spacing. Hence, the current technology for portal frames significantly underutilises the structural capability of sandwich panel systems. While portal frames offer the highest structural efficiency for the frame, the current construction technology does not necessarily offer the highest structural efficiency for the frame and envelope together.

Exploiting the enhanced structural capabilities offered by modern sandwich panels with increased insulation levels may present a major opportunity for the supporting structure, which can be reengineered to account for the envelope's increased strength and stiffness in order be lighter or with reduced part counts and consequently with reduced embodied carbon. This would require an examination of the options which offer highest structural efficiency for the frame and building envelope together.

Consideration should also be given to the current practice for design and specification of industrial buildings, which requires the designers for the structure and cladding to work in separate. If the cladding is required to act as a main structural element in additional to fulfilling its conventional roles, structural and cladding designers would be required to work in unison. Although this would be likely to be a step change from the current procurement practice and would add more stringent requirements for the cladding in terms of design, procurement and construction management, it would be likely to be justifiable if greater efficiency is achieved.

The following potential opportunities were identified in terms of exploiting the structural capability arising from the increased insulation depth in sandwich panel envelope systems:

1. Increasing the span of the sandwich panel roof systems and reducing the number of structural members within building.
2. Exploiting the in-plane strength and stiffness of sandwich panels arising from the increased cross-sectional area and the stiff insulation and utilising diaphragm action for building stability and stiffness.
3. Exploiting the out-of-plane and in-plane (axial and shear) strength of sandwich panels to engineer them as primary structure and remove primary frame elements for frameless construction of small buildings.

A review of the current state of the art and earlier work associated with each of those opportunities is provided in the following sections within the current chapter:

- Section 2.4: Long span building envelope systems
- Section 2.5: Buildings with diaphragm action
- Section 2.6: Frameless buildings

Each of the opportunities has been shortlisted for further investigation and the results are provided and discussed in detail in Chapter 4, Chapter 5 and Chapter 6.

2.4 Long span building envelope systems

Long span building envelope systems spanning directly between primary frame components and eliminating the requirement for secondary structure has been an appealing idea to the single-storey industrial building industry for some long time. The use of long span envelope systems may:

- Reduce the part counts on site and consequently, the installation time.
- Reduce the number of joints to the supporting structure and consequently, the length of air-leakage and thermal bridging sources, theoretically leading to benefits in terms of energy conservation during the in-use phase of the building.

- Allow the frames to be spaced further apart, consequently reducing the amount of frames within the building and the associated construction cost in terms of material and part counts.

There are primarily three systems currently used for long-span applications:

- Structural liner tray systems for walls.
- Structural decking systems with membranes for roofs
- Sandwich panel systems for walls

An overview of the systems is provided herein.

2.4.1 Structural liner trays

An illustration of the structural liner tray system is shown in Figure 2.9. Structural liner trays comprise deep structural profiles typically between 90mm-150mm with rigid insulation slabs placed within the profile's depth on site. The system is finished with external profile sheeting. Due to the depth of the profile, the heavy profiling to increase the sheets' s stiffness and strength and the increased steel gauge (between 0.75mm-1.25mm), structural liner trays possess adequate strength and bending moment resistance to span directly between columns, removing the requirement for side rails.

The system provides advantages against speed of construction when compared to built-up systems requiring side rails. However, there are significant problems associated with thermal bridging, which are overcome by placing additional insulation outside the depth of the liner tray, consequently leading overuse of materials. Furthermore, while liner trays can provide restraint to the outer flange of the columns, where restraint to the inner flange of columns is required such as for plastically designed portal frames, the absence of side rails is problematic. This is because 'stays' cannot be attached to the liner trays easily. This is another reason why structural liner trays are rarely used as roof cladding particularly for portal frame construction. For these reasons, the system is not met commonly in the UK; however it is popular in Germany and elsewhere, where frames are not plastically designed and is common practice to add more insulation to address the thermal issues.

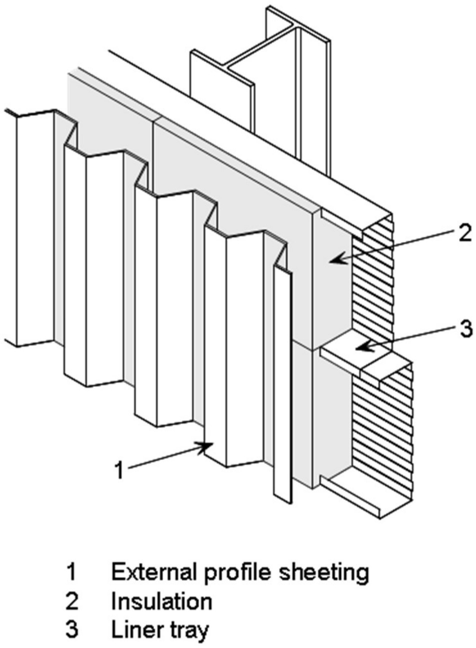


Figure 2.9 Structural liner tray system; Source: steelconstruction.info

2.4.2 Structural decking with membrane

An illustration of the structural decking and membrane system is shown Figure 2.10. Structural deck and membrane systems are long span roof systems providing an alternative to built-up cladding. They are mainly used where low-pitch or flat roof is required, and they are often used for big flat trussed-roof buildings. They comprise trapezoidal metal deck with sufficient depth (varying from as low as 32mm up to 210mm in extreme cases) and gauge (typically from 0.70mm up to 1.20mm) in order to span directly between beams or trusses. Rigid insulation is placed on site on top of the deck and the system is finished with a waterproof membrane.

The system can provide restraint to the outer flange of the roof beams or truss chords; however, similarly to structural liner trays, they are not suitable to provide restraint to the inner flange since knee-bracing cannot be easily attached. Hence, the system is also largely unsuitable for portal frames where there are onerous restraining requirements. Also, the membrane used as an external layer has much lower life span compared to the steel outer skins with high quality coatings; hence maintenance and replacement costs need to be considered.

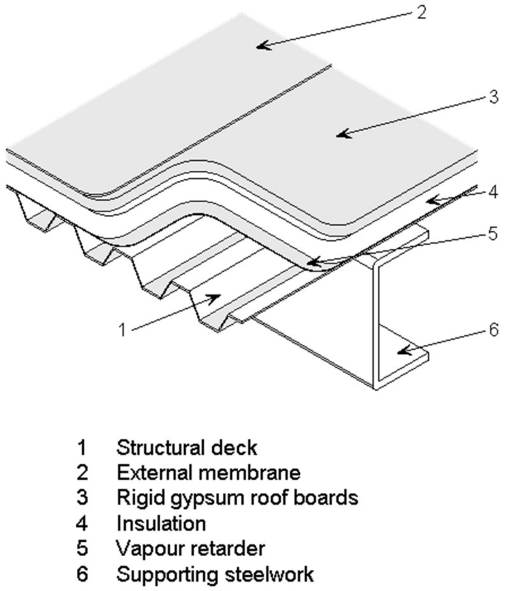


Figure 2.10 Structural decking system; Source: steelconstruction.info

A variation of the structural decking system with the commercial name X-Dek is available in the market by Kingspan to be used as a roof application (Kingspan, 2014). The system comprises a deep inner steel decking layer infilled with self-expanding PIR foam insulation, topped with an additional insulation layer and finished with an outer layer of a flat metal sheet or membrane system. The system can span up to 8.0m for typical loads; however it is not suitable for pitched roofs because of the flat outer skin (whether flat metal of membrane). As for conventional structural decking, the system suffers from lack of provisions for attachment of 'stays' to provide restraints to the rafters; hence, it remains largely unsuitable for portal frames. Also, the system is much deeper when compared to sandwich panels systems for similar thermal performance. Some exploitation of the insulation applies in terms of providing restraint to the decking plates and preventing premature buckling of the sheets, i.e. improved use of materials is made compared to the plain decking systems. Nevertheless, the system demonstrates merely no structural exploitation of the increased insulation depth.

2.4.3 Long-span sandwich panel walls

In practice, sandwich panel wall systems are the only building envelope types widely used for long span applications. Sandwich panel wall systems comprise either PIR or mineral wool insulation and can span directly between columns, eliminating the need for secondary structure. As for sandwich panels in general, the system provides excellent thermal performance, hence it is far superior to the structural liner trays for long span wall applications. Air-tight joints with provisions for minimising thermal bridging and thermal loss are applied. The panels can provide restraint to the outer flange of the columns if needed. Due to absence of cladding rails, attachment of 'stays' to the panels and the inner flange of the columns is very difficult. Hence, the system works well with any type of frame without very onerous restraining requirements for the inner flange of the columns, such as trussed-roof systems, simple construction or elastically-designed portal frames.

Table 2.7 and Table 2.8 demonstrate the achievable spans associated with the reduced U-values and increased insulation depths of two typical sandwich panels systems, utilising PIR (with lightly profiled faces) and Mineral Wool (with flat faces) insulation cores respectively. An illustration of typical wall panel cross-sections and the definition of insulation thickness is shown in Figure 2.11. The data demonstrate that long spans up to 9.5m (for single-span conditions) can be achieved with current panel specifications when the increased insulation depth is exploited for structural purposes. The information is only indicative and based on manufacturer's data, assuming typical wind values of 0.8kN/m^2 in pressure and 0.5kN/m^2 in suction and calculation according to established codes of practice for the specification of sandwich panels (BS EN 14509:2013).

Table 2.7 Span capabilities of typical wall sandwich panels with PIR insulation

U-value	**Insulation (PIR) thickness**	**Application**	**Span (single)**
0.31W/m^{2}K	70mm	Part L 2010 backstop	4.4m
0.24W/m^{2}K	90mm	Part L 2010 notional	5.3m
0.17W/m^{2}K	120mm		6.5m

Outer sheet: micro-ribbed 0.7mm; Inner sheet: lightly profiled 0.4mm; S220 steel grade; for full panel specifications refer to Appendix A.

Table 2.8 Span capabilities of typical wall sandwich panels with mineral wool insulation

U-value	**Insulation (MW) thickness**	**Application**	**Span (single)**
0.31W/m^{2}K	125mm	Part L 2010 backstop	6.9m
0.26W/m^{2}K	150mm	Part L 2010 notional	7.5m
0.22W/m^{2}K	175mm		8.1m
0.19W/m^{2}K	200mm		8.7m
0.17W/m2K	240mm		9.5m

Inner sheet: flat 0.7mm; Inner sheet: flat 0.5mm; S220 steel grade; for full panel specifications refer to Appendix A.

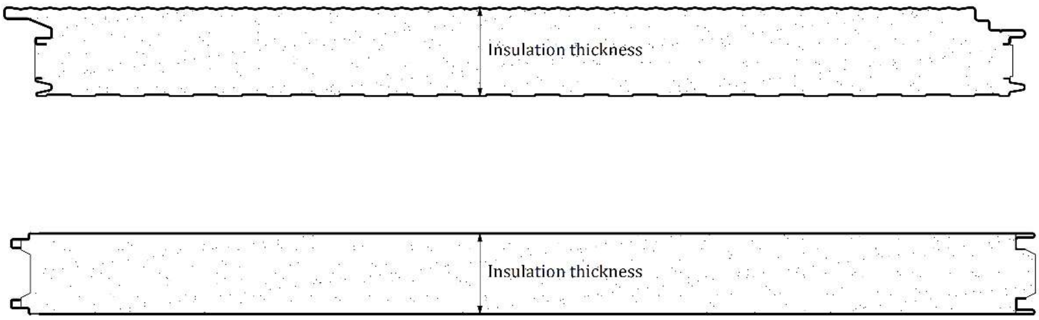


Figure 2.11 Typical cross sections and definition of insulation thickness for wall sandwich panels with lightly profiled faces and flat faces

The governing failure modes for the estimated long spans are typically compressive failure of the outer face in pressure or the inner face in suction or deflections. Achievement of the spans is dependent on the support width to be wide enough to prevent compressive failure of the core in wind pressure and adequate number of fasteners to prevent pull-out in wind suction.

2.4.4 Long-span sandwich panel roofs

As noted in Section 2.1.4.2, modern sandwich panels systems possess increased stiffness and load-carrying capacity due to their increased insulation depth for Part L compliance. Table 2.9 shows the achievable spans for a roof sandwich panel with fully profiled outer sheet and a lightly profiled inner sheet. An illustration of typical roof panel cross-section and the definition of insulation thickness is shown in Figure 2.12. The data demonstrate that modern roof sandwich panels with increased insulation thicknesses to accommodate Part L requirements can span up to 6.6m. The information is only indicative and based on manufacturer's data, assuming typical imposed load of 0.6kN/m^2 and 0.8kN/m^2 for wind suction and calculation according to established codes of practice for the specification of sandwich panels (BS EN 14509:2013). It is, therefore, evident that the current structural capability of profiled roof sandwich panels is significantly underutilised when panels are required to span up to typically 1.8m to 2.0m. Further modifications and Part L revisions would be likely to allow for roof sandwich panels to span even further.

Table 2.9 Span capabilities of typical roof sandwich panels

U-value	Insulation (PIR) thickness	Application	Span (single)
0.25W/m^{2}K	80mm	Part L 2010 backstop	4.7m
0.20W/m^{2}K	100mm		5.4m
0.16W/m^{2}K	120mm	Part L 2010 notional	6.1m
0.15W/m^{2}K	135mm		6.6m

Outer sheet: fully profiled 0.5mm; Inner sheet: lightly profiled 0.4mm; S220 steel grade; for full panel specifications refer to Appendix A.

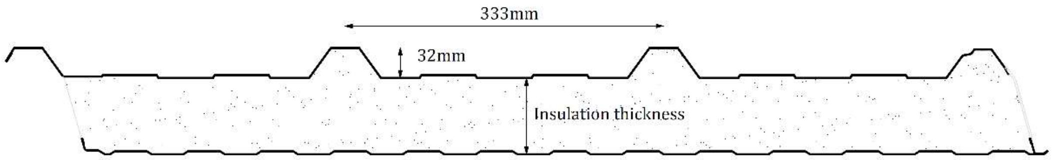


Figure 2.12 Typical cross section and definition of insulation thickness for roof sandwich panels with a fully profiled and a lightly profiled face

The load bearing capacity of modern roof sandwich panels at long spans is typically limited by the compressive resistance of the sheets, being yielding failure of fully profiled sheets or flexural wrinkling failure of lightly profiled sheets. These depend on the depth and stiffness of the profiles, the strength and gauge of steel, as well as on the stiffness of the core which stabilises the sheets against premature failure when under axial compression. Consequently, improvements to the geometrical and mechanical properties of the steel sheets and cores create an opportunity to improve the load-carrying capacity of the panels. Further discussion will be given in Chapter 7.

Despite the evident capabilities, sandwich panel roof systems for long span applications have so far been little examined and the focus has mainly been on the capability of the panels alone, without consideration of arrangements for the supporting structure. The system has been originally engineered to span between purlins with an orientation from ridge to eaves to allow for the rain water to run off. The orientation of the profiles in the span direction would not suit spanning directly between rafters or the top chords of trusses without the use of an additional layer which would allow the rain water to still flow from ridge to eaves. Hence, a holistic review of the system together with the supporting structure would be required, if opportunities for long-span roof panels were to be advanced. A review of the previous work undertaken for long span sandwich panels is presented herein.

2.4.4.1 Previous work

Berner (2008) and Berner *et. al.* (2009) examined the use of flat sandwich panel system spanning directly between rafters of duo-pitched frames as long-span envelopes. The use of flat outer sheet permits the panels to span directly between the primary frame members, eliminating the problem of corrugations. The study focused on the creep behaviour of the PUR insulation core, i.e. the loss of its stiffness over time under it permanent load. This would have an effect on the long-term stiffness of the panels, as well as on the stabilising function of the flat sheets. It was demonstrated that even with reduction of the insulation core's stiffness, the panels would still be able to accommodate long spans. The author also looked at various water-tight joints to be applied at the seams and end joints of adjacent panels. The flat panels would suffer from water leakage if appropriate detailing is not applied. This is a problem which profiled panels spanning from ridge to eaves would not suffer. The study comprised experimental investigations of joint arrangements and demonstrated that with appropriate detailing water-tight joints can be achieved. Use of flat panels spanning directly between frames and possessing water-tight joints would eliminate the need for over cladding. However, for typical applications that would require insulation thicknesses beyond those required for thermal requirements alone. Also, the study did not include any considerations regarding any benefits or impacts of the long span sandwich panel use on the frame.

Berner (2010) examined the use of curved sandwich panels with fully profiled outer sheets to achieve long spans and various architectural possibilities. The author examined curved panels spanning between flat beams and rafters, overcoming rainwater drainage issues due to the curve geometry of the roof. It was demonstrated that by exploiting the arch action, bending moments can be practically negligible and all the forces can be resisted by the in-plane axial resistance of the panel. The significant reduction of bending moments allows the panel to span further. A 6.0m arch radius was considered throughout the study, reflecting the capability of an existing manufacturer. The study also discussed the manufacturing requirements and limitations, particularly with regards applied radius in the continuous production and the height of arch, as well as provisions for the support connections and arrangements. Despite the demonstration of long span capabilities, engineering of arched panels would pose very challenging production issues and significant alterations and additions to the current continuous flat panel production lines. In addition, there is currently no evidence of scope for curved roof applications; hence it would be unlikely that investment would be meaningful in this field, particularly when the capabilities of conventional products have not been thoroughly examined. The increased arch height may increase the internal volume of buildings, and consequently the energy demand for heating. Finally, a curved roof would not work with the highly efficient pitched portal frames, and the scope would solely rely on frame with flat rafters / beams, hence not the most efficient forms of construction.

Heywood and Moutaftsis (2012c) at the SCI examined the feasibility of developing long span sandwich panels for roof applications to achieve similar spans as decking systems with membranes. The study was based on testing of conventional sandwich panels and modern analytical methods to theoretically quantify the stiffness and resistance of long span panels. The feasibility studies demonstrated that fully profiled panels with increased profiled depths or micro-ribbed panels with increased insulation thickness can achieve very large spans. Efficient modifications to the steel profiles in terms of geometry and gauging were estimated. Consideration was also given to the orientation of the profiles with regards to the frame and the requirement for rain water run-off. An over-cladding system with halter and standing seam sheeting was also examined.

Similarly, Moutaftsis and Heywood (2011) at the SCI examined the opportunities of using high strength steel for roof and wall sandwich panels. Their study showed that using high strength steel for fully profiled sandwich panels can permit increased spans to be achieved or reduction of the outer sheet's steel gauge for the current typical span requirements if reduced material use is sought.

Kurpiela (2013) and Kurpriela and Lange (2013) carried out a cost-optimisation study on lightly profiled sandwich wall panels using PUR insulation and steel sheeting. The study comprised analysing test data to obtain material properties, deriving relationships between the density of PUR cores and its mechanical properties and suggesting improvements to the design methods for the compression resistance of the steel sheets. A Pareto-optimisation exercise was undertaken with cost and span being two competing objectives. The study defined sets of Pareto-optimal solutions as well as a single solution for various cost scenarios. Each solution corresponded to a set of panel properties able to achieve a specific maximum span for a defined set of applied loads. Within the context of the particular exercise, large spans at approximately 5m were derived as cost-optimal. Similarly, Studzinski *et. al.* (2009) undertook an optimisation study for lightly profiled sandwich panels with steel faces and PUR cores aiming to minimise cost and maximise span, using evolutionary algorithms. Within the context of the given study it was found that economically viable panel arrangements were found for spans up to 4.5m. However, the optimisation studies by Kurpiela (2013), Kurpriela and Lange (2013) and Studzinski *et. al.* (2009) were focused on the panel elements alone and did not take into account any benefits or implications on the spacing of the supporting structural members. Also, the studies did not take into account considerations regarding the thermal requirements, which influence the insulation thickness.

2.4.4.2 Opportunities

All the previous studies focused on the feasibility of specifying, constructing and implementing long span sandwich panels. There is now little doubt that long span sandwich panels can be achieved with the increased insulation levels currently required in Part L for thermal purposes. Spanning opportunities may be even greater if the trend of insulation levels to increase is materialised in the future. Furthermore, improvements in the core and steel sheet mechanical and geometrical properties may create further opportunities for enhancing load-bearing capacity. However, none of the earlier studies focused on the structural benefit of using long span building envelopes, nor how structural forms could best accommodate long span sandwich panel systems.

Current sandwich panel systems work very well at short spans for portal frames in collaboration with purlins and provisions for attachments of 'stays' to provide the required restraints to the portal frames, as explained in Sections 2.1.3, 2.1.4 and 2.1.5. Hence, it is difficult to apply long-span roof envelopes with the current state of the art. Additionally, with portal frames being the predominant form of construction and yielding high structural efficiency for the frame, there is little scope for long span sandwich panels to permit higher structural efficiency for the frame itself.

However, allowing for alternative structural forms which would best accommodate long span sandwich panels systems with modern insulation levels and exploit their existing structural capability may permit a higher structural efficiency for the frame and envelope together.

The long span opportunity was further examined and the results of the study are given in Chapter 4. Long-span roof sandwich panels are further examined in Chapter 8.

2.5 Buildings with diaphragm action

Diaphragm action in single-storey buildings, often referred to as 'stressed skin' action, concerns the cladding's inherent in-plane strength and stiffness and its effect on the strength, stiffness and stability of the structural frame. The cladding can provide an alternative load path so that the loads applied externally to the building can be distributed between framing and cladding. This results in a reduction of moments and forces in the clad frame compared to the case of bare frame. Stressed skin design takes into account the in-plane strength and stiffness of the cladding to design more efficient frames.

Stressed skin design is widely used in design and construction of aircraft, ships, cars and trains. For buildings, it was first appreciated during 1950's, when the measured stresses and deflections of various single-storey steel-framed buildings with their cladding installed were found to be significantly less than the theoretical values of bare frames (Bryan and El-Dakhakhni, 1964, Bates and Bryan, 1965, Bryan, 1971, Davies, 1973, Bryan and Davies, 1975). Since then, stressed skin design largely evolved during '70s and '80s (Bryan and Davies, 1975, Bryan, 1976) and was ultimately incorporated in manuals and design standards (Davies and Bryan, 1984, BS 5950-9:1994, ECCS, 1995, BS EN 1993-1-3:2006). However, stressed skin design has not been widely used in the UK for the reasons which are explained in Section 2.5.1.

2.5.1 Design objectives

The cladding provides additional stiffness to the frame, corresponding to deflections and forces which are reduced to those calculated for bare frames. Although this may have a positive effect in design, particularly when deflections govern, the stiffness of the cladding does not always prove conservative. This is because stiff cladding attracts higher forces; hence it is possible that stiff skins become overstressed, even at working loads. Consequently, this may have a damaging effect on the cladding.

The interaction between cladding and framing occurs whether the building is designed for this or not. Designers often take notional account of the reduction of the frame deflections due to the stiffness of the cladding without performing calculations. The design standard for steel-framed buildings BS EN 1993-1-1:2006, makes allowance for the stiffness provided by the cladding to the bare frame in the calculation of the stability parameter α_{cr}. Similarly, the deflection limits for portal frames recommended by the SCI Advisory Desk Note AD 090 are differentiated between different types of cladding.

Modern design methods included in design standards allow the designers to design frames and cladding safely and achieve structural economy by using diaphragm action to:

- Eliminate or reduce bracing (particularly horizontal)
- Reduce deflections due to frame spread under vertical loads

- Reduce deflections due to frame sway under horizontal loads
- Reduce the forces in the primary structure, and hence to reduce the steel weight
- Provide lateral stability to beams and rafters
- Produce 'frameless' buildings (more discussion on this aspect is given in Section 2.6).

For buildings designed for diaphragm action, the cladding is required to act as a main structural element, in addition to fulfilling its conventional roles. Consequently, achieving greater efficiency for frames comes with additional and more stringent requirements for the cladding in terms of design, procurement and construction management. If the envelope is required to act as primary structure the cladding contractor and structural engineer or steelwork contractor would need closer coordination and the structural function of the roof should be taken into account during the design the structure accordingly. Removal or replacement of the cladding through the life of the building would need to be avoided or carefully designed. Within the current industry framework, structure and envelope are typically specified by different parties, while the cladding is typically specified after the structure has been fully designed. Consequently, use of stressed skin design would require a reconsideration of the current procurement practice which would be a significant step change for the single-storey industrial buildings sector.

Furthermore, diaphragm action is weakened by the presence of openings. Effects on diaphragm action from openings in the roof of less than 3% of the roof area may be disregarded (BS 5950-9:1994). However, modern industrial buildings typically possess openings at approximately 12% of the roof area for natural lighting and energy conservation purposes according to Part L. Consequently, making use of diaphragm action requires improvements in the roof envelope to compensate for loss of strength and stiffness due to increased opening areas.

These two limitations are the main reasons why stressed skin design has not been extensively used in the UK.

2.5.2 Design standards and manuals

The current state-of-the-art relies on the relevant British Standard 'Code of practice for stressed skin design' BS 5950-9:1994 and the 'European recommendations for the application of metal sheeting acting as diaphragm – Stressed skin design' (ECCS, 1995). The latter practically reflects to BS 5950-9:1994 with a few additions of provisions for structural liner trays and stabilisation of hot-rolled members by sheeting. Provisions for diaphragm action have been adopted in the Eurocode BS EN 1993-1-3:2006 which supersedes BS 5950-9:1994. However, the Eurocode standard is rather immature and brief in guidance and refers to ECCS (1995) for further information. An important contribution of the BS EN 1993-1-3:2006 is the inclusion of provisions for structural liner tray systems (often referred to as 'cassettes') which are not included in the earlier BSI (1994) and are briefly examined in the ECCS (1995) documents. More recently, Davies (2006) highlighted deficiencies of design expressions in both BS 5950-9:1994 and ECCS (1995).

The aforementioned standards practically reflect and refer to the 'Manual of stressed skin diaphragm design' by Davies and Bryan (1984) which is considered a de facto standard itself. Another comprehensive manual on the subject is the publication by Höglund (2002), written for the Swedish Institute of Steel Construction (SBI).

2.5.3 Principles of structural behaviour

Davies and Bryan (1984) have previously described the basic concept of diaphragm action: The application of vertical gravity loading to single-storey pitched-roof portal frame structures causes the apex to deflect downwards and the eaves to spread outwards and in opposite directions. This movement is usually named as 'spread' and causes the building envelope system to stress in order to comply with the frame deflections while at the same time deploying its inherent in-plane stiffness to resist these deformations. Similarly, when a frame with either pitched or flat roof is subjected to lateral horizontal loads, the external forces cause the eaves to 'sway', i.e. deflect laterally and to the direction of loading. Similar to the earlier case, the deflection compatibility requires the envelope to be subjected to stresses, while on the same time it tends to resist those deflections due to its inherent in-plane stiffness. These two modes form the basis of diaphragm action, i.e. the ability of the envelope to resist in-plane loading.

For single-storey framed structures under stressed skin action, the roof may be regarded as a deep plate girder spanning between the gables. Under in-plane load, the sheeting acts as the web of a beam, the gables as reaction supports and the edge members as beam flanges in axial tension and compression. This also specifies the load path to the ground, since diaphragm forces are transferred through the roof and the frames to the vertically braced gables and then to the foundations.

A distinction may be made between two cases (see Figure 2.13):

- For flat roofs with diaphragm action, the lateral load applied on the structure is causing the frame to 'sway'. The roof acts to resist against that 'sway' mode, relying on its in-plane stiffness and resistance. No diaphragm contribution is being made for load acting vertically. In order for the diaphragm action to be effective in the 'sway' mode, the gable frames need to be adequately braced in order to prevent movement and resist the end reactions occurring from the beam analogy (Davies and Bryan, 1984).
- For pitched frames, the in-plane load to the diaphragm occurs (1) through the lateral load acting to the structure, causing the eaves to deflect and the frame to move in a 'sway' mode and also (2) through the vertical roof load which causes the eaves to 'spread'. This makes the diaphragm potentially effective for both 'sway' and 'spread' loading, with the latter one being highly dependent on the roof pitch (the higher the pitch the greater the diaphragm's contribution in resisting 'spread' loading). In order for the diaphragm to act against 'spread' modes, the gable frames need to be braced at the eaves level; when the gable frames are fully braced, contribution to 'sway' is also made (Davies and Bryan, 1984), making the assembly potentially effective at both modes.

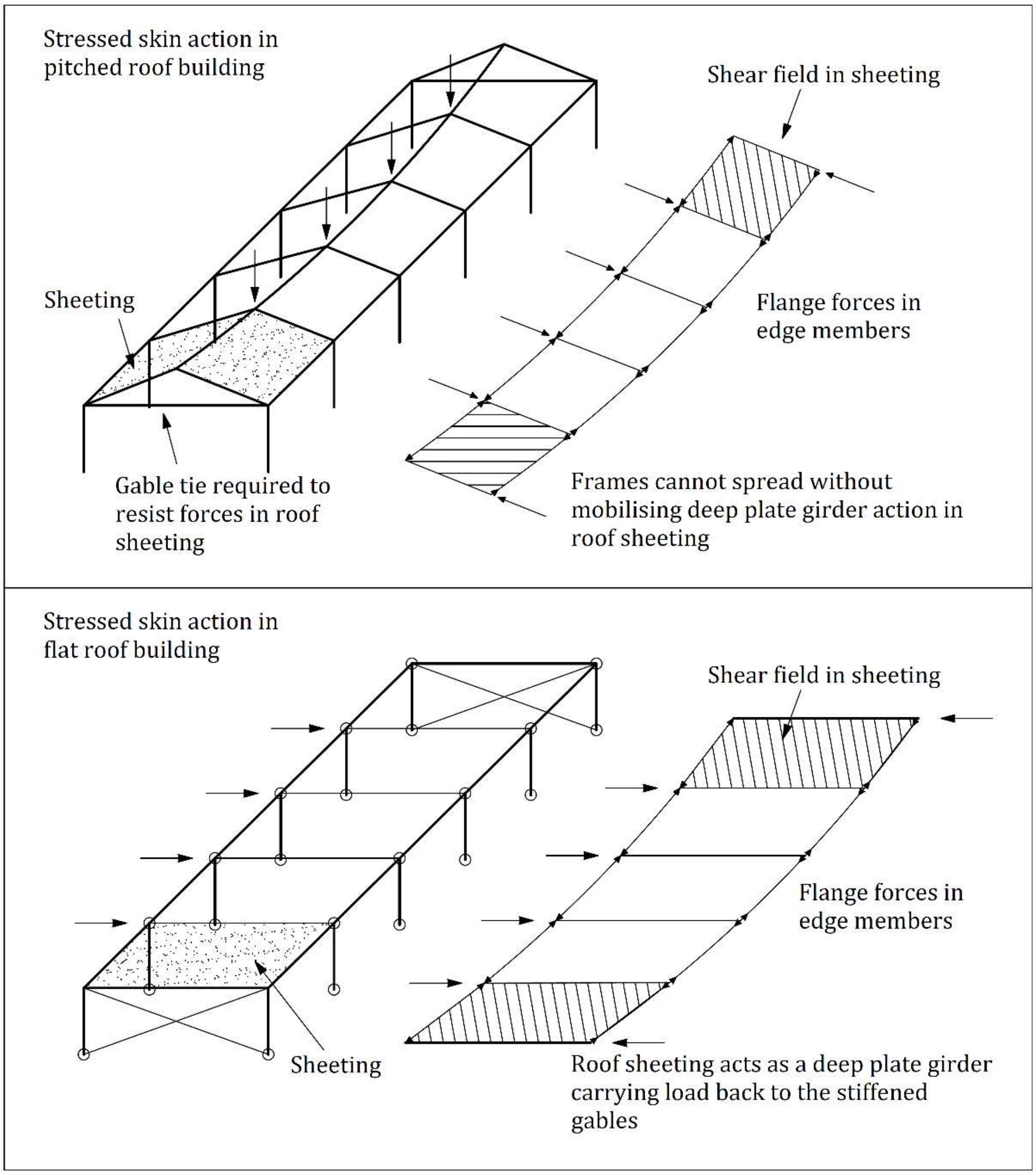


Figure 2.13 Stressed skin action in buildings (adapted from BS 5950-9:1994)

The effectiveness of the diaphragm action of the roof is dependent on the end reactions (shear) being adequately resisted by the end bays; this requires that the edge frames are virtually immovable. This is achieved by providing stiff braced gable frames which is the common practice.

According to BS 5950-9:1994 a more significant effect of diaphragm action can be present when:

- For flat-roof buildings
    - under horizontal load: the length / width ratio is less than 4.0.
- For pitched-roof buildings

- under vertical load: the length / width ratio is less than 2.0
- under horizontal load: the length / width ratio is less than 4.0
- for resisting vertical load: the roof pitch is higher than 10^{o}
- when horizontal load is only applied to one or two frames.

For buildings with a series of frames matching the 'beam' analogy (such as industrial sheds with a series of portal frames), shear forces and deformations generally dominate over the bending ones, due to the proportions of the roof (Davies and Bryan, 1984). However, bending (axial) stresses to the edge members are also considered important and therefore, need to be taken into account.

Diaphragm roof systems may be effective on pinned-jointed frames, acting, in that case, on their own against the side load to the structure and resisting the sum of the in-plane loading, hence the stability of the structure relies on the diaphragm action (BS 5950-9:1994). This is very common for low-rise flat roofed structures with 'simple' construction (pinned-jointed connections between primary components) (Davies and Bryan, 1984). The other alternative scenario is when the diaphragm roof is installed on rigid-jointed frames, where the in-plane load is being shared between the frames and the diaphragm (BS 5950-9:1994). Davies and Bryan (1984) listed a number of advantageous cases for utilisation of diaphragm action in conjunction with rigid frames, most of them primarily relying on cases being dominated by side deflections. For portal frames, the utility of the diaphragm action is highly dependent on the roof pitch since this dictates what if the proportion of the vertical load resisted by the diaphragm is considerable or not, while the resistance to the 'sway' deflections is anticipated to be important. The load distribution between frames and diaphragms will be discussed in detail in Chapter 5.

The resistance and stiffness of a diaphragm relies on the components types and numbers used, as well as in the fastening arrangements and the span direction. This is further discussed in Chapter 5.

2.5.4 Early research on full clad buildings

Early tests of fully sheeted buildings were performed in the 1960's and 1970's before standards and manuals were developed.

Bryan and El-Dakhakhni (1964) tested portal frames with steep roofs (30^{o}) when either bare comprising purlins or sheeted with corrugated steel cladding and under 'spread' loading. The tests showed that significant bending moment reduction occurred for the sheeted building by a minimum of 34% for the pinned base cases and 24% for fixed bases, demonstrating that for a stiffer frame the relieving effects of the sheeting were reduced. The failure load of the sheeted frame was approximately 42% higher than the failure load of the bare frame, demonstrating the significant diaphragm action contribution of the sheet.

Bates *et. al.* (1965) tested for 'spread' loading a relatively large portal frame structure (of approximately 50m span) comprising a steep roof pitch of 12^{o} and aluminium sheeting with roof-light arrangements. Large moment reduction was shown for the sheeted building compared to the bare frame and reduction of deflections at approximately 50%, demonstrating the large contribution of diaphragm action in terms of strength and stiffness under 'spread' load.

Bryan (1971) showed that steel sheeting mounted on purlins of a 7m span flat roof frame lead to significant reduction of stresses and deflections by over 43%, when under a sway load. Also ductile shear diaphragms allowed for much larger load to be resisted at ultimate failure in comparison to non-ductile.

2.5.5 Modern envelope systems

All the current standards and manuals discussed in Section 2.5.2 envisage that the main cladding applications for diaphragm action are trapezoidal decking systems or structural liner trays in ECCS (1995) and BS EN 1993-1-3:2006, fastened to purlins or rafters with screws or pins to the crests or troughs.

Davies and Lawson (1999) examined the diaphragm action of modern roof cladding systems, including twin-skin built-up systems with and without spacers, structural liner trays, standing seams and composite panels with through and secret fixings. The intention of the research was to examine the capabilities for modern (at that time) envelope systems which are engineered to achieve higher insulation levels and often reduced number of fasteners for air-tightness and moisture-prevention requirements, which may reduce the diaphragm action capabilities. The research included a mixture of experimental and analytical studies, based on tests performed by Davies and Deakin (1993). Davies and Bryan (1984) was the main literature source used for the analysis.

The study showed that good diaphragm strength and stiffness can be achieved for built-up systems and structural liner trays (cassettes), while standing seams showed little capability. Built-up systems were found to make best use of composite action in-plane between the outer and liner sheet, similarly to two springs in parallel. The sandwich panel systems demonstrated excellent stiffness but weakness in strength due to premature tearing of the thin inner (liner) sheet and further discussion is given in the following section.

Since then, focus has mostly been paid to diaphragm action of structural liner trays (cassettes), largely investigated by Davies (2002, 2004, 2006) and based on earlier work of Baehre and Buca (1986), Baehre (1987), Baehre et al. (1990). However, cassette systems are rarely used in the UK for the reasons already discussed in Section 2.4.1.

Mahendran (1994) experimentally investigated crest-fixed profiled cladding acting as diaphragm together with resisting wind forces. The study showed that in-plane shear forces have an insignificant effect on the static or cyclic wind uplift resistance.

2.5.6 Sandwich panel diaphragms

While decking and structural liner tray systems have been thoroughly examined during the past decades, there has been significantly less research for sandwich panels systems despite their significant inherent component stiffness and strength. Similarly, the use of sandwich panels for diaphragm action is outside the strict scope of the current design standards. Recent research advances in sandwich panel technology evaluated the diaphragm action behaviour and opportunities for sandwich panels systems. These are discussed herein.

The study by Davies and Lawson (1999) examined the diaphragm action of two sandwich panel types and arrangements: sandwich panels with through-fixings at 475mm centres along the panel edges; and sandwich panels with 'secret' fixings at 280mm centres along the purlins. The test results and analysis demonstrated that sandwich panels are excellent systems in terms of stiffness and strength; however they suffer from premature failure at the fixings, hence not providing sufficient strength. In particular, the thin inner (liner) sheet was found to tear due to the eccentricity of the force applied to the end fastener.

Although the results were not promising, there were a few parameters which differ to current practice and may lead to reconsideration whether sandwich panels are suitable for diaphragm action applications. In particular, through-fixings were spaced at larger distances than those typically used in modern practice (at typically 300mm or 250mm centres). More fasteners tend to increase the shear strength of the system. Also, none of the systems comprised seam fasteners which are typical in practice for weather-tightness purposes. Seam-fastened systems lead to a different force distribution compared to systems without them, particularly attracting a proportion of the shear forces and relieving the end fasteners. Finally, secret fixings do not provide any restraint to longitudinal movement, hence it is reasonable to assume that these are not suitable for diaphragm action, while they are not anymore used anyway.

It is also worth mentioning that no failure mode associated with loss of stability or failure of the profiled sheets was recorded. This is reasonable considering the effective stabilising effect of the core to the sheets.

Overall, modern sandwich panel arrangements with seam fasteners and more dense edge spacing may prove more suitable for diaphragm action applications either as used in practice or with further enhancements.

Mahendran and Subaaharan (2002) carried out an experimental investigation of basic crest-fixed sandwich panel diaphragms fastened on 3m spaced purlins without use of shear connectors. Improvements in fastening systems were proposed, comprising increased numbers of end fastener rows at every crest together with increased spacing of seam fasteners. Optimum arrangements were found to yield considerable strength and stiffness compared to conventional arrangements. The authors also proposed a new analytical model which was validated against the test results. The crest-fixed practice is common in Australia but not the UK, where valley-fixed arrangements are typically adopted. The research did not examine the effect of the improved sandwich panel diaphragms on full structures.

The project with the name EASIE (Ensuring Advancement in Sandwich Construction through Innovation and Exploitation) was undertaken between 2010 and 2012 to investigate opportunities for frameless buildings made by sandwich panels. Diaphragm action aspects of sandwich panels were investigated using mixture of experimental and analytical studies. In particular:

Käpplein and Ummenhofer (2011) developed an improved model for the calculation of stiffness and strength for end and seam fasteners used for sandwich panels. The development of the model was based on experimental (Käpplein and Misiek, 2010b) and analytical research (Käpplein and Misiek, 2011b) and comprised fastening provisions on either cold-formed components (such as purlins or cladding rails) or hot-rolled components, for attachment of sandwich panels directly on the primary frame.

Käpplein *et. al.* (2012) investigated the stabilising effect of sandwich panels on their supporting elements and developed a model for the calculation of the in-plane shear strength and stiffness of the panels. The study was based on experimental and analytical investigations in Käpplein and Misiek (2010a) and Käpplein and Misiek (2011a) respectively and was also discussed in Käpplein *et. al.* (2010).

Dürr *et. al.* (2011) investigated the torsional restraint provided by sandwich panels to beams against lateral torsional buckling. The study incorporated experimental and numerical research and a new model for the calculation of rotational stiffness of beams stabilised by sandwich panels was developed.

The design models developed by Käpplein and Ummenhofer (2011), Käpplein *et. al.* (2012) and Dürr *et. al.* (2011) in the aforementioned studies are incorporated in the 'European Recommendations on the Stabilisation of Steel Structures by Sandwich Panels' (CIB-ECCS, 2013). The document provides guidance for the design of steel members by taking into account the stabilising effect (lateral or torsional) of sandwich panels.

2.5.7 Recent research on clad framed buildings

Mahendran (1997) carried out full-scale tests on portal frames to examine the effects of diaphragm action and also compare against 3-dimensional numerical modelling results. The experimental studies were on a tall, low roof pitch frame, with length / width ratio equal to 1 (12m x 12m in plan). The frame was examined when bare and clad with roof and wall trapezoidal sheeting under gravity and lateral loads. The experimental results showed that for the lateral load case a 33% reduction of bending moments and 66% reduction of deflections occurs in the clad frame compared to the bare case. Little differences were shown under gravity load due to the low pitch. However, the study was not entirely clear which load combination was dominant for design and considering the tall height it would be likely that lateral loads could govern. A full 3-dimensional model was then developed to examine the potential effects of using sandwich panel diaphragms as well as the elimination of roof bracing and incorporate them in a cost comparison against the conventional clad portal frame design. Equivalent bracing members were used to account for the panel stiffness and strength. The analysis showed a 10% reduction of the frame cost with the aid of sandwich panels. The overall cost of the building was found to be higher due to the cost of the sandwich panel units, however a remark was made of the additional benefits in terms of energy conservation.

Gurung and Mahendran (2002) carried out a comparative life costs assessment between a conventionally designed frame using single sheeting and a stressed-skin designed frame with sandwich panels. The building length / width ratio was very small, 1.44 (36m x 25m). A 3-dimensional analysis was used to determine member sizes. The study showed that a more cost efficient primary frame may be achieved when the diaphragm action of sandwich panels is exploited compared to the conventional design. The study examined the costs in terms of frame, cladding and operational energy holistically and significant savings with the use of sandwich panels were demonstrated. The operational energy performance of the building with sandwich panels was based on Jayasinghe and Mahendran (2003).

Phan *et. al.* (2015) investigated numerically the use of stressed skin action for cold-formed portal frames with semi-rigid connections when using trapezoidal sheeting with purlins. All the modelled frames were 12m span, 3m high to eaves, with 10^{o} roof pitch. The length of the building, hence aspect ratio, was varied, also the spacing between frames. A 3-dimesional analysis using spring elements was undertaken according to BS 5950-9:1994 and incorporated into an optimisation algorithm to identify weight-optimal assemblies. The results showed that significant reduction of frame material, up to 53%, can be achieved when exploiting diaphragm action. The reduction was greater for buildings with smaller length/width ratio, i.e. when diaphragm action contribution is most significant. It should also be highlighted that cold formed and semi-rigid frames are fairly flexible by definition, hence the contribution of diaphragm action is deemed to be significant. The roof pitch of 10^{o} was such that allowed reduction of movement and bending moments for the frame under vertical load, to the point that the governing load case shifted from wind to gravity.

Nagy *et. al.* (2016) numerically examined the effect of stressed skin trapezoidal sheeting with Z-purlins on the stability of hot-rolled portal frames. The examined frames had a relatively high building length (90m) to width ratio (2x12m double-span frames), typical frame spacing (6m) and 8^{o} roof pitch. Various diaphragm arrangements were evaluated for their impact to the frame. The analysis showed that diaphragm action was particularly effective in reducing the sway-sensitivity of the frame and increasing the load multiplication factor a_{cr}.

DeMateis and Landolfo (2000a, 200b) examined the behaviour of sandwich panel diaphragm roofs and walls under cyclic loading and their use in seismic applications for steel single-storey trussed-roof industrial buildings. The studies demonstrated steelwork savings up to 20%, which is reasonable considering the effectiveness of diaphragm under ‘sway’ loads.

2.5.8 Opportunities

The previous studies on clad portal frames focused on:

- Frames with relatively high-pitched roofs, which are ideal for the exploitation of diaphragm action under spread loads which typically govern the design of buildings
- Buildings with small length / width ratio which are also ideal for the exploitation of diaphragm action
- Buildings which are relatively tall and narrow, hence dominated by sway loads for which diaphragm action is largely effective.

The results showed excellent scope in terms of deflection and frame forces reduction under the above circumstances. However, recent developments in best practice for single-storey buildings may require that the opportunities are re-examined. In specific:

Current best practice incorporates the use of, primarily, low pitch roofs, typically at 6^{o}. This is for reduction of building volume and operational energy requirements in terms of heating or cooling. Additionally, requirements in the recent revisions of Part L of the Building Regulations are that increased areas of rooflights are adopted to permit use of natural light. Presence of openings within diaphragms over 3% of roof area reduces the strength and stiffness of the diaphragms while requiring strengthening around the opening edges. These requirements may limit the opportunities of exploiting diaphragm action.

On the other hand, higher grades of steel are increasingly being used in the UK, leading to higher strength but increased flexibility of the frames. Diaphragm action has a higher contribution in flexible frames, hence, there is an opportunity where the benefits of diaphragm action may be exploited.

Moreover, sandwich panels are still an ideal means of diaphragm action in terms of component strength and stiffness and previous studies showed that improvements in terms of fastening are possible, while failure modes associated with the sheeting itself are largely avoided due to the stiffening effect of the insulation core. Also, it is very common that sandwich roof panels are seam-fastened for weather-tightness purposes, which provides increased strength and stiffness. In addition, recent developments in theory provide opportunities for a more accurate estimation of fastener strength and stiffness which can allow a reliable parametric investigation. Furthermore, long-span opportunities due to increased insulation thickness allow sandwich panels to be used in arrangements spanning in parallel to the diaphragms, an opportunity which has not been previously examined.

Finally, the use of the building envelope as primary structure would be likely to require reconsideration of the current procurements framework and a closer coordination between the designers for the structure and envelope.

The opportunity for buildings with diaphragm action using sandwich panels was further examined and the results of the study are given in Chapter 5.

2.6 Frameless buildings

The concept of utilising the building envelope to act as primary structure and eliminate or reduce the primary frame in buildings has been appealing to the industry and researchers. The routes of the concept are linked to those for shipping, aeronautic and airspace industries, where frameless structures with stiffened envelope are very frequent and designed to resist the applied loads. In construction, small ancillary buildings utilising stiff self-supported metal envelopes are common for farming and military applications; however there has been hardly much use for larger applications.

Using the envelope to construct frameless buildings may be achieved on the basis of:

- Roof components acting as roof diaphragm for strength and stiffness under in-plane shear loading, together with resisting lateral out-of-plane loading as conventionally.
- Wall components acting as columns/walls under vertical axial loading and wall diaphragms for strength and stiffness under in-plane shear loading, together with resisting the lateral out-of-plane loading as conventionally.
- Both wall and roof systems acting together with appropriate connection arrangements to provide stability to the building structure.

Frameless construction would be likely to reduce or eliminate the structure within the building and consequently the associated construction materials, part counts, installation time and ultimately cost and embodied carbon.

There are primarily three systems which have been either used or researched in the past for frameless building construction:

- Light-gauge folded plate roofs

- Structural liner trays
- Sandwich panels

Folded plate roofs and structural liner trays are now obsolete and their use in frameless construction has been minimal for reasons which will be explained in the next sections. Sandwich panels are already used in cold stores as small internal structures; hence their use for frameless construction has been limited. However, recent research has shown promising capabilities for larger frameless structures. An overview of the systems is provided herein.

2.6.1 Folded plate roofs

One of the earliest design methods for elimination of primary structure for roofs was with the aid of 'folded plate roof' construction. This method of construction comprised large heavily profiled light gauge steel sheets connected together and designed to resist both in-plane and out-of-plane loading through diaphragm action (Bryan, 1976). Profiled members at the fold lines along the ridge, apex and valley were required for connections, while gable framing and sheeted walls completed the structural system. An illustration of the system is shown in Figure 2.14.

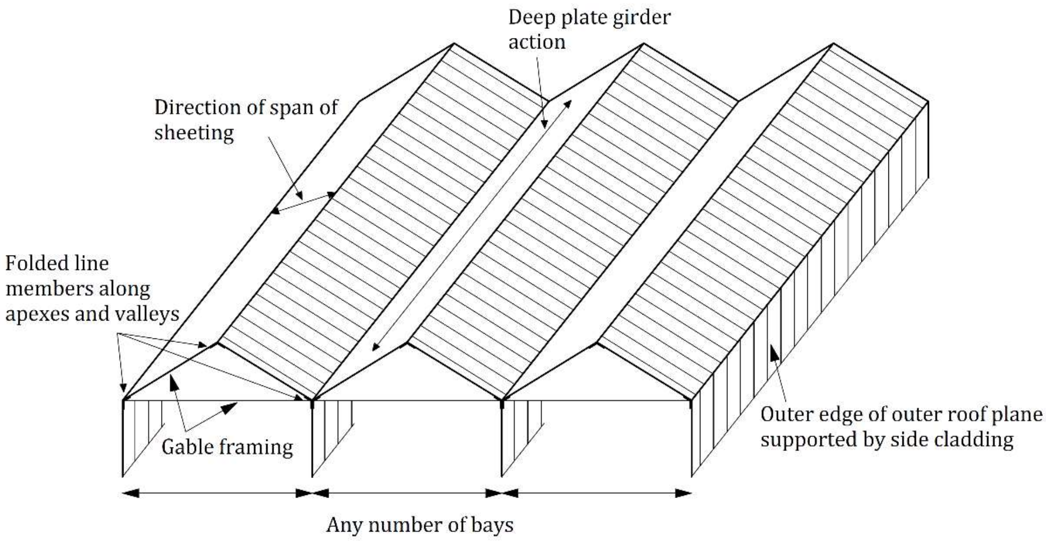


Figure 2.14 General arrangement of folded plate roof (adapted from BS 5950-9:1994)

Although this method of construction was not strictly frameless for the whole building, as columns were required as normal, while stability was provided by either bracings or sheeted wall diaphragms, it eliminated the need for any primary structure within the roof.

The structural behaviour of the system was largely similar to diaphragm action and design guidance on the same principles as for stressed skin design was developed by Davies (1976) and incorporated in the manual by Davies and Bryan (1974). The British Standard for stressed skin design BS 5950-9:1994 includes the most up-to-date design guidance for folded plate roofs.

Similar to stressed skin design, the stability of the system relied on the diaphragm action of the roof, which plays the role of primary structure. Vertical and horizontal loading into the roof create a load component in the plane of the roof slope. For this in-plane load, each roof slope was designed as a deep girder spanning between gables, with the shear forces in the girder's web carried by the folded plate roof diaphragm and the axial tension and compression forces by the folded line members. The reaction of the girder as transferred to the gable ends which are required to be tied. As well as resisting in-plane shear forces, the folded plate of the roof also resisted the out-of-plane loading by spanning directly between the fold line members and in turn, providing to them lateral and torsional restraint. Columns are required at least at the gable corners, but intermediate fames can be eliminated depending on the roof girder span between the gables. The columns and the side sheeting resist the vertical load. In the absence of vertical bracing, the side sheeting can also work as a wall diaphragm (BS 5950-9:1994).

The system has been hardly used in the UK for reasons similar to those as for the diaphragm action method explained in Section 2.5.1, regarding the envelope acting as primary structure, procurement and effects of openings. Furthermore, while conventional stressed skin design can be performed with the aid of conventional cladding systems, folded plate roofs require a heavy engineering for the roof sheets in comparison to conventional cladding systems which is hardly justifiable unless the method is widely used.

2.6.2 Frameless buildings using structural liner trays

More recently, Davies (2006) examined the opportunities of using structural liner trays (light gauge cassettes) as self-supporting wall systems to resist in-plane shear forces through stressed skin action and axial loads from roof and the stories above, together with out-of-plane wind loading. The study showed that for low-rise buildings, with single or a small number of stories, structural liner trays can be used to eliminate primary frame. As building examples, a house and a clinic in France were mentioned, together with use of the system for modular construction. Davies (2009) highlighted the high structural capability of structural liner rays in terms of diaphragm, axial and bending resistance and showed conceptual detailing arrangements. The critical effects of openings on the shear resistance of the envelope were discussed. Also, opportunities for seismic applications were presented, where liner trays can be used to resist cyclic in-plane shear loads, while offering a lightweight structure, consequently reducing the seismic forces.

As already highlighted in Section 2.4.1, structural liner trays are rarely used in new construction in the UK and possess serious thermal bridging issues which can require additional insulation. Also, their structural capabilities primarily rely on the resistance of the skins, sometimes also to the restraint provided by rigid insulation, but not due to insulation thickness. Hence, there is generally little scope to make use of the insulation for structural purposes.

2.6.3 Frameless buildings using sandwich panels.

Sandwich panels are currently used as roof and wall cladding elements supported by secondary or primary structure. They are designed to resist primarily out-of-plane loads (wind, snow imposed), as well as thermal loads due to the temperature difference at the inner and outer sheets arising from the insulating function of the core. Their very high core stiffness and the stabilising action against in-plane distortion of the sheets, make them ideal means to resist in-plane shear forces. As for long-span applications, their high bending moment resistance arising from the profile geometries and insulation depth, provides not only increased capacities against out-of-plane loading, but potentially also for bending moments and forces arising from in-plane axial loads. These enhanced structural capabilities are likely to create opportunities for frameless buildings where entire or part of the primary structure is replaced by sandwich panels.

Frameless structures applications with sandwich panels already exist in cold-stores (Davies, 2001). These are typically relatively small internal structures, resisting very small structural loads, primarily their self-weight, as well as thermal stresses due to temperature differences. These applications are very limited.

Significant research has been recently undertaken to examine the opportunities of constructing frameless buildings with the aid of sandwich panels. Two major projects were carried out on this field with the project names EASIE (Ensuring Advancement in Sandwich Construction through Innovation and Exploitation) at DarmStadt and SandSet at iS-Mainz. These are discussed herein.

2.6.3.1 EASIE project

Käpplein and Misiek (2010c) examined the use of sandwich panels acting as columns and resisting vertical loads and performed a series of tests on axially loaded sandwich panels to investigate their global axial resistance. Flat and lightly profiled panels were examined, comprising steel or Glass Fibre Reinforced Plastics (GFRP) sheets and PUR or EPS insulation cores. The panels were storey-high (2.5 to 3.5m). Global imperfections in the form of initial out-of-plane deflections were introduced during testing and failure modes to the ultimate load were recorded. Load introduction was engineered such that failure of the connections at the supports was avoided. Failure modes comprised compressive failure of the sheets or shear failure of the core near the supports. Käpplein and Misiek (2010d) experimentally examined the effects of creep on axially loaded sandwich panels.

Käpplein and Misiek (2011d) examined analytically and numerically the global resistance of axially loaded sandwich panels against the test results of Käpplein and Misiek (2010c) and Käpplein and Misiek (2010d). The authors proposed two alternative design methods for the calculation of stresses and the quantification of the model ultimate panel resistance under axial loads: these were a) according to the second order theory, with an amplification factor due to second order effects applied to the calculated stresses; and b) according to the equivalent member design, using equivalent member buckling curves and axial and bending moment load combination formulas based on the shape of the bending moment diagram. Both methods were found to yield very close results in excellent agreement with the numerical model. Finally, design guidance for the treatment of creep was also offered.

Käpplein and Misiek (2011e) examined the introduction of load into axially loaded sandwich panels experimentally and numerically. The test results showed that connections would be susceptible to crippling failure and are the limiting factor of the axial load introduced to axially panels. Crippling is, similar to wrinkling, a stability (buckling) failure mode for a sheet elastically supported by the core material and subjected to an axial compressive (normal) force. Crippling occurs when the loaded edge of the sheet is unsupported in the thickness direction, hence the force is introduced into a free edge. Consequently, crippling is noticed in sandwich panels at the axial load introduction areas. Crippling is slightly different that wrinkling which occurs when the loaded edge of the sheet is supported in the direction and rotations are restrained, hence occurring at the span of sandwich panels. Edge imperfections, such as those associated with panel cutting can have very onerous effects to the crippling resistance. Significant differences were noted in the test results depending on whether forces are introduced to one or both panel sheets of the axially loaded panels. A design method for the quantification of the crippling resistance of the sandwich panel sheets was also given, using an equivalent member buckling approach for the steel sheets subject in compression.

Käpplein and Misiek (2010b) carried out tests to determine the shear resistance and flexibility of edge and seam fasteners for sandwich panels and Käpplein and Misiek (2011b) offered analytical design guidance.

Käpplein, S., Misiek, T. (2011f) developed a numerical and analytical model to calculate the stress and stiffness distribution in sandwich panel diaphragms for simple frameless buildings, i.e. without substructure, with the exception of the members used for connections between roofs and walls. The model considered seam-fastened roof panels and wall panels with and without seam fasteners. The connection was considered as those typical for small internal frameless buildings in cold-stores. It is worth noting that the panels were considered infinitely stiff and the shear response was modelled based purely on the stiffness of the fasteners. The concept of the model was largely similar to the concept for stressed skin design (Davies, 1973, BS 950-9:1994) although a comparison between the methods was not made.

The design guidance from Käpplein and Misiek (2011d, 2011e, 2011f) is summarised in the EASIE Design Guidance (Käpplein and Misiek, 2011g) and calculation examples are also available by Käpplein and Misiek (2011h). Furthermore, concise guidance based on the EASIE Design Guidance is available – Käpplein and Ummenhofer (2010) for sandwich panels under axial load and load introduction, and Käpplein and Ummenhofer (2011) for the shear resistance and flexibilities of sandwich panel connections. The latter is also included in the recent European Recommendations document by CIB-ECCS (2013).

The output of the EASIE project was very useful in terms of developing analytical design guidance and demonstrating the capability of modern sandwich panels to resist loads required for frameless buildings. However, the research did not mature enough to examine the feasibility of frameless buildings holistically, neither to identify the associated benefits and limitations or further technical barriers, such as effects of openings in shear resistance and stability, or use of panels as girders around openings.

2.6.3.2 SandSet project

Naujoks and Hörnel-Metzger (2013) at iS-Mainz carried out an extensive research programme to investigate the opportunities for sandwich panels used as self-supported load-bearing structural elements.

The study examined experimentally and numerically the resistance and stress distribution of lightly profiled sandwich panels under axial load, the corresponding modes of failure and effects of creep. A numerical model was also developed and the formula from EASIE (Käpplein and Misiek, 2011d) was validated.

Connections between vertical sandwich panels used as walls / columns and horizontal sandwich panels used as floors were also investigated. Self-tapping and through-fastened connections were examined, while consideration was also giving in minimising thermal bridging effects. The load distribution between the inner and outer sheets of the vertical panels was examined, showing that all the connections are suitable to transfer forces and the load distribution being dependant on the connection type. Nevertheless, design guidance on this aspect was not developed.

The racking strength and stiffness of wall sandwich panels when used as shear walls in a vertical orientation were also examined. The panels where fastened to a steel base replicating fixed support conditions. Various arrangements were examined in terms of number of fasteners at the base of the panel and seams. Failure modes were examined and a numerical model was also validated. A design method was also proposed for the particular arrangement. The results were also presented and discussed by Lange *et. al.* (2011).

Berner (2009) utilised data shown in the SandSet publications in order to investigate the possibilities of designing frameless small and medium size house buildings with sandwich panels. The study comprised a typical house layout of a 8m x 11m footprint, with one or two stories and a soffit. For the applied wind loads on the buildings, it was shown that sandwich panels used as primary structural members were adequate. The panels were examined acting as columns, shear walls, girders around openings and also as floor element in the intermediate floors.

2.6.3.3 Opportunities

Recent research on frameless buildings made of sandwich panels has produced analytical guidance which now allows sandwich panels to be designed as structural members replacing primary structure, acting as strut-columns and shear diaphragm walls and roofs. Effects of creep, load introduction into the panel sheets and reliable estimation of connection resistances and flexibilities are also included in the recent literature. While more research would still be required, particularly in terms of effects of openings in shear diaphragms and provisions for connections, the current literature allows developing design concepts for frameless buildings with sandwich panels.

Wall and roof panels with modern specifications already comprise deep insulations for energy conservation purposes according to Part L. This allows high bending resistances and stiffness to be achieved when panels are in out-of-plane load as well as under in-plane axial load and second order effects. Similarly, the stiff cores prevent shear buckling and profiled distortion of the sheets when panels act as shear diaphragms. Moreover, the compressive resistance of the sheets can be exploited not only to resist out-of-plane loads, but also in-plane axial loads when panels act as strut-columns and at the connections where loads are introduced. Finally, while up to date research has so far focused on small buildings, such as cold-stores, the long span capabilities of modern roof sandwich panels creates opportunities to produce buildings of larger sizes.

Until now there has been very little research in examining the feasibility of frameless buildings with sandwich panels in larger sizes or investigating the overall benefits to the structure in terms of material usage or identifying and addressing further technical and commercial barriers. Hence, an opportunity exists to exploit the modern panel specifications and insulation levels for energy conservation purposes and use the existing literature to investigate the range of applicability of sandwich panels for frameless buildings, evaluate the barriers and quantify the associated benefit in terms of structure, cost and embodied carbon.

The opportunity for frameless buildings using sandwich panels was further examined and the results of the study are given in Chapter 6.

Chapter 3 Research methodology

Following the review of the state-of-the-art for single storey industrial buildings, the related carbon emissions aspects and the review of the opportunities, the current chapter describes the research methodology adopted to address the aims and objectives outlined in Chapter 1 with respect to the opportunities identified in Chapter 2.

3.1 Methodology

The adopted methodology is described in Sections 3.1.1 to 3.1.3. The outline and flow chart are presented in Section 3.1.4.

3.1.1 Feasibility studies

3.1.1.1 Preliminary studies on current and enhanced structural capabilities of sandwich panel envelope technologies

The options for exploiting the inherent material of sandwich panel envelope systems have been investigated. The following opportunities were selected:

- Long span roof cladding.
- Diaphragm action.
- Replacement of primary structural components.

For each opportunity an analytical study was carried out to:

- Determine the potential of the current sandwich panel envelope technology to deliver enhanced structural capabilities.
- Identify the scope for engineering of novel envelope solutions with enhanced structural capabilities and quantify their performance.

Sandwich panel products with modern specifications were used as a base case and a full list of specifications is included in Appendix A. The structural behaviour of sandwich panels was examined based on established modelling methods, principles for sandwich structures and current specialist literature. In particular, the structural modelling was based on the procedures according to:

- The design standard BS EN 14509:2013 and Davies *et al.* (2001) for panel arrangements under out-of-plane loading.

- The design standards BS 5950-9:1994, CIB-ECCS (2013) and the design guidelines from EASIE (Käpplein and Misiek , 2011g) and Käpplein and Ummenhofer (2011) for panel arrangements under in-plane shear loading.
- The design guideline from EASIE (Käpplein and Misiek, 2011g) and Käpplein and Ummenhofer (2010) for panel arrangements under in-plane vertical loading.

The panel resistance checks were according to:

- Annex E in BS EN 14509 for panels under out-of-plane loading.
- Section 4 and 5 in BS 5950-9:1994 and Section 3 in CIB-ECCS (2013) for panels under in-plane shear loading.
- Section 7 in the EASIE design guideline (Käpplein and Misiek, 2011g) for panels under in-plane vertical loading.

The material properties for the sheets and cores were obtained by the panel manufacturer. The compressive resistances of the sandwich panel sheets (local buckling or flexural wrinkling) used for the feasibility studies were obtained from the panel manufacturer, while those used for the study discussed in Chapter 7 were derived by testing and analysis according to the procedures of BS EN 14509:2013. The compressive resistances of the sheets at the load application areas were determined according to Section 8 in the EASIE guideline (Käpplein and Misiek, 2011b).

Resistances of fasteners under out-of-plane loading were based on typical and widely used products and were according to manufacturer’s datasheets (a full list of specifications is included in Appendix A). Resistances of fastening arrangements under in-plane loading were calculated according to the model of Käpplein and Ummenhofer (2011) adopted in CIB-ECCS (2013).

The task identified the potential of making use of the inherent strength and stiffness of sandwich insulated envelope systems to produce more efficient structures when compared to the current practice.

3.1.1.2 Determination of building structure – envelope assembly forms

In order to maximise the use of the envelope’s capability, the supporting frame structure might need to be reconfigured. For that purpose, alternative structural frame arrangements were investigated and solutions were conceptually designed such that they accommodate the features of the various envelope schemes and exploit their enhanced structural capabilities.

The primary areas of focus were:

- Building forms which accommodate the long span capabilities of the envelope, including provisions for frame restraint.
- Building forms which exploit the diaphragm action capabilities of the envelope.
- Frameless and semi-frameless buildings.

Three typical warehouse building sizes (small, medium, and large) were chosen for the long span and diaphragm action studies in order to enable comparisons with previous work undertaken at the Architectural Engineering Research Group at Oxford Brookes University. These sizes are considered typical and sufficiently representative in the UK. For the frameless buildings study, the building sizes were limited by the spanning capability of modern roof sandwich panel systems, as shown later. The details of the chosen building geometries are given in Chapter 4, Chapter 5 and Chapter 6.

Modelling of each structural scheme was carried out using two-dimensional (2-D) elastic analysis. It was assumed that all frames within each building were the same. This is representative of typical consulting engineers' practice. An engineering analysis was performed by a mixture of hand calculations and commercial structural modelling software following the standards explained below.

The frames (portal and truss-roofed) were designed according to BS EN 1993-1-1:2005 and the relevant UK National Annex. Principles from BS 5950-1:2001 which do not contradict BS EN 1993-1-1:2005 were also used, together with the guidance of Koschmidder and Brown (2012), Brown (2013) and Salter *et al*. (2004) for portal frames. Frame stability checks were performed according to Clause 5.2 in BS EN 1993-1-1:2005. The column and beam member checks were according to the following clauses in BS EN 1993-1-1:2005:

- Clause 6.2 for cross-sectional resistance checks.
- Clause 6.3 for member buckling resistance checks.

For the diaphragm action study, BS 5950-9:1994 was used to model the coupled behaviour of envelope diaphragm and frames. The use of these standards particularly concerned the distribution of forces between the envelope and the frame (Section 7 in BS 5950-9:1994), as well as verification of the cladding under in-plane shear (Sections 5 and 6 in BS 5950-9:1994). The frames were designed according to BS EN 1993-1-1:2005 using the aforementioned beam, column and frame stability checks.

The analysis of the frameless buildings was performed based on the guidance mentioned in Section 3.1.1.1 for sandwich panels under in-plane (vertical and shear) and out-of-plane loading. The stability of frames was assessed by developing an appropriate non-linear model based on engineering principles and appropriate for hand calculations.

Details of the modelling methods applied for each case are explained in Chapter 4, Chapter 5 and Chapter 6.

A comparative study was carried out, including a series of analyses for the various building sizes and for combinations of the alternative structural forms and the envelope systems with enhanced structural capabilities. The structural performance of the schemes were compared against the base case and the structural efficiency was determined in terms of frame weight.

The technical barriers that prevent holistic design of the envelope solutions and supporting structure acting in unison were evaluated and conceptual engineering design principles were applied to overcome these limitations. Where potential solutions were identified, these were shortlisted to be investigated in further depth to determine their feasibility.

The advantages and disadvantages of each envelope-structure configuration were, finally, evaluated and discussed.

The output of the task was the recommendation of optimised reconfigured structural forms to accommodate sandwich panel envelope systems with enhanced structural capabilities for various building sizes.

Based on the results from 3.1.1.1 and 3.1.1.2 a decision was made whether each opportunity would be studied further. The opportunities which were found likely to yield the greatest benefit and their identified technical barriers were shortlisted to be further researched. The detailed decision-making process is shown in Section 3.4.

3.1.2 Addressing of technology barriers

The identified scope for engineering of novel sandwich panel solutions (Section 3.1.1.1) and shortlisted technical barriers (Section 3.1.1.2) for the selected building opportunities were addressed through further research in order to provide technically feasible solutions and quantify associated embodied carbon increases if applicable. This part of the research may be categorised into two different areas:

1. Proposing sandwich panel solutions to accommodate the enhanced structural requirements for the selected building forms through:
    a) Assessing the impact of material properties reliability and design guidance conservatism on the modelling of structural performance.
    b) Design of improved sandwich panel solutions, making use of current guidance and test data and through application of design optimisation methods.
2. Evaluate the performance of the systems in terms of embodied carbon.

All the proposed engineering solutions were approached on the basis of combined analytical investigation and experimental work. The details are included in Chapter 7.

The output of the task was the proposal for optimised re-engineered sandwich panels and the demonstration of the feasibility of their structural performance to allow their use with the selected reconfigured structural forms.

3.1.3 Review systems in terms of embodied carbon and cost

For the selected opportunities and structural schemes and based on the results of the structural appraisal and technology barriers' investigation, the structure-envelope assemblies were holistically reviewed in terms of embodied carbon and relative cost.

3.1.3.1 Embodied carbon appraisal

The building arrangements (structure-envelope assemblies) were appraised for their embodied carbon emissions and a comparison with the impact of the conventional building technology was undertaken. Due to the complexity of embodied carbon quantification methods, the lack of a standard method of measurement and the uncertainties associated with end-of-life options, a choice of the most appropriate method was required. Since the nature of the study was comparative, the embodied carbon emissions were calculated using the established Life Cycle Inventory (LCI) database Inventory of Carbon and Energy (ICE) by the University of Bath (Jones and Hammond, 2008), which includes cradle-to-gate system boundaries and does not require the use of specialist software. The methodology used is described in more detail in Chapter 7 and Chapter 8.

3.1.3.2 Cost appraisal

A comparative cost analysis was carried out for the structure-envelope assemblies to assess the relative cost options of the selected structure-envelope assemblies against conventional building technology based on material quantities (weight of steel) and construction rates. Each structural option was evaluated based on mean values of materials, fabrication and erection costs. The cost analysis guidance data which may be found in the Spon's Architects' and Builders' Price Book 2015 (AECOM, 2015) were used in the appraisal. Complimentary updated guidance data from industry liaison were used where appropriate. The methodology used is described in more detail in Chapter 8.

3.1.4 Research outline and flow chart

The research methodology is outlined below:

1) Define a series of typical building sizes (see Section 3.2).
2) For each building size:
    a) Define a base scheme for the structural frame to reflect the current construction practice and a series of re-engineered structural frame schemes to accommodate the opportunities for exploiting the structural capability of the cladding systems, in terms of:
        i. long span
        ii. diaphragm action
        iii. frameless construction
    b) Identify the loading on the structure for each scheme (See Section 3.3)
    c) Model the frames based on established codes of practice and current literature; identify technical barriers and apply engineering principles to overcome limitations
    d) Assess the structural capability of the envelope for each opportunity and couple with the frame modelling if necessary
    e) Design the structural frames for each scheme.
3) Appraise the structural efficiency of all schemes in terms of frame weight.
4) Discuss the advantages and disadvantages of each scheme and shortlist their associated technology barriers.
5) Decide whether to advance each opportunity and scheme for further research and exploitation based on a decision-making process (see Section 3.4).
6) For the selected opportunities and schemes, propose sandwich panel solutions to accommodate the enhanced structural requirements for the selected building forms through analytical and experimental work to:
    a) Assess the impact of material properties reliability and design guidance conservatism on the modelling of structural performance

   b) Design of improved sandwich panel solutions, making use of current guidance and test data and through application of design optimisation methods.
   c) Evaluate the performance of the systems in terms of embodied carbon.

7) Review the selected envelope – structural forms assemblies in terms of:
   a) Embodied carbon, based on established Life Cycle Inventory databases, reflecting the used envelope forms and considering the optimum combination of envelope and structure for the chosen buildings.
   b) Relative cost, based on calculated component and construction rates.

A flow chart of the research outline is shown in Figure 3.1.

The detailed methodology and modelling methods for each set of aforementioned studies is discussed in each relevant chapter. The input and assumptions with regards to the actions on structures (Section 3.1.4 (2b)) which are relevant throughout the individual research steps are presented in Section 3.2. The decision-making process whether to advance each opportunity and scheme is shown in Section 3.4.

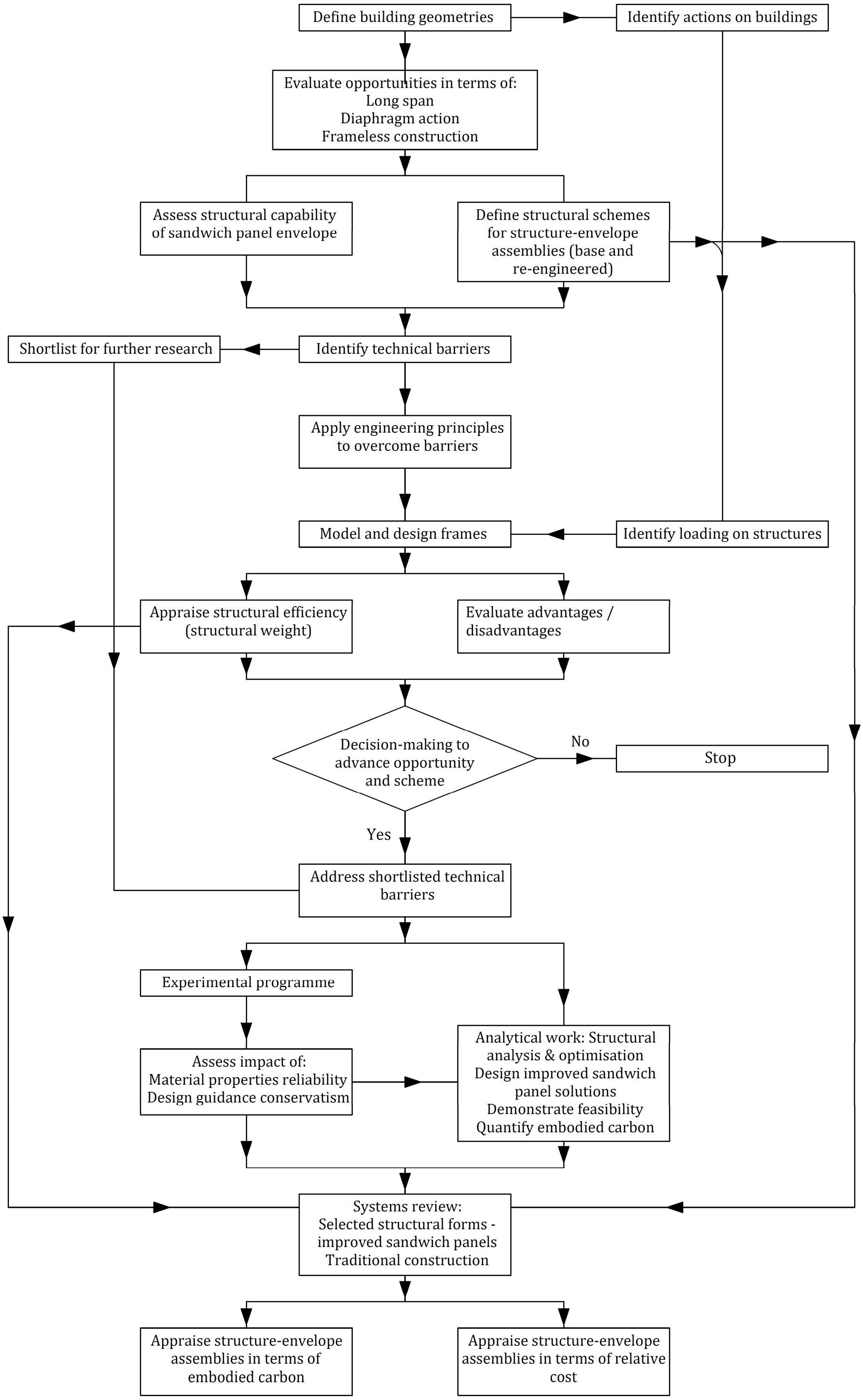


Figure 3.1 Flow chart of research outline

3.2 Actions on buildings

The actions and loads used in the structural analysis were obtained from the following design standards and their corresponding National Annexes for the UK:

- BS EN 1990:2002 for combinations of actions
- BS EN 1991-1-1:2002 for permanent and imposed loads
- BS EN 1991-1-3:2003 for wind loads
- BS EN 1991-1-4:2005 for snow loads

The load magnitudes for each building, their derivation and the combinations of actions for Ultimate Limit States (ULS) and Serviceability Limit States are shown in Appendix B in the following sections:

- B.1 Permanent load
- B.2 Imposed load
- B.3 Snow load
- B.4 Wind load
- B.5 Combinations of actions

Combinations of actions occurring together were considered according to expression 6.10 of BS EN 1990:2002 and the relevant UK National Annex, together with the partial and combination factors included in the standard.

Temperature gradients on the building envelope were according to BS EN 14509:2013.

3.3 Decision-making process

For each of the identified opportunities and schemes and based on the results of the structural appraisal, a decision-making process was applied to decide whether to advance the opportunity through further research (see Section 3.1.1 (5)). The process was based on assessing the impact of the following features for each opportunity and structural scheme:

1) Scope for frame material reduction
2) Range of building sizes for which the opportunity is applicable
3) Technical barriers and required further research
4) Level of envelope or structure re-engineering required in order to address the identified technical barriers and step-change from current practice.

5) Advantages and disadvantages, particularly concerning technical limitations, construction features and potential required workmanship

The decision-making process is illustrated in Figure 3.2. Only the selected opportunities were carried forward for further research after the completion of the feasibility studies.

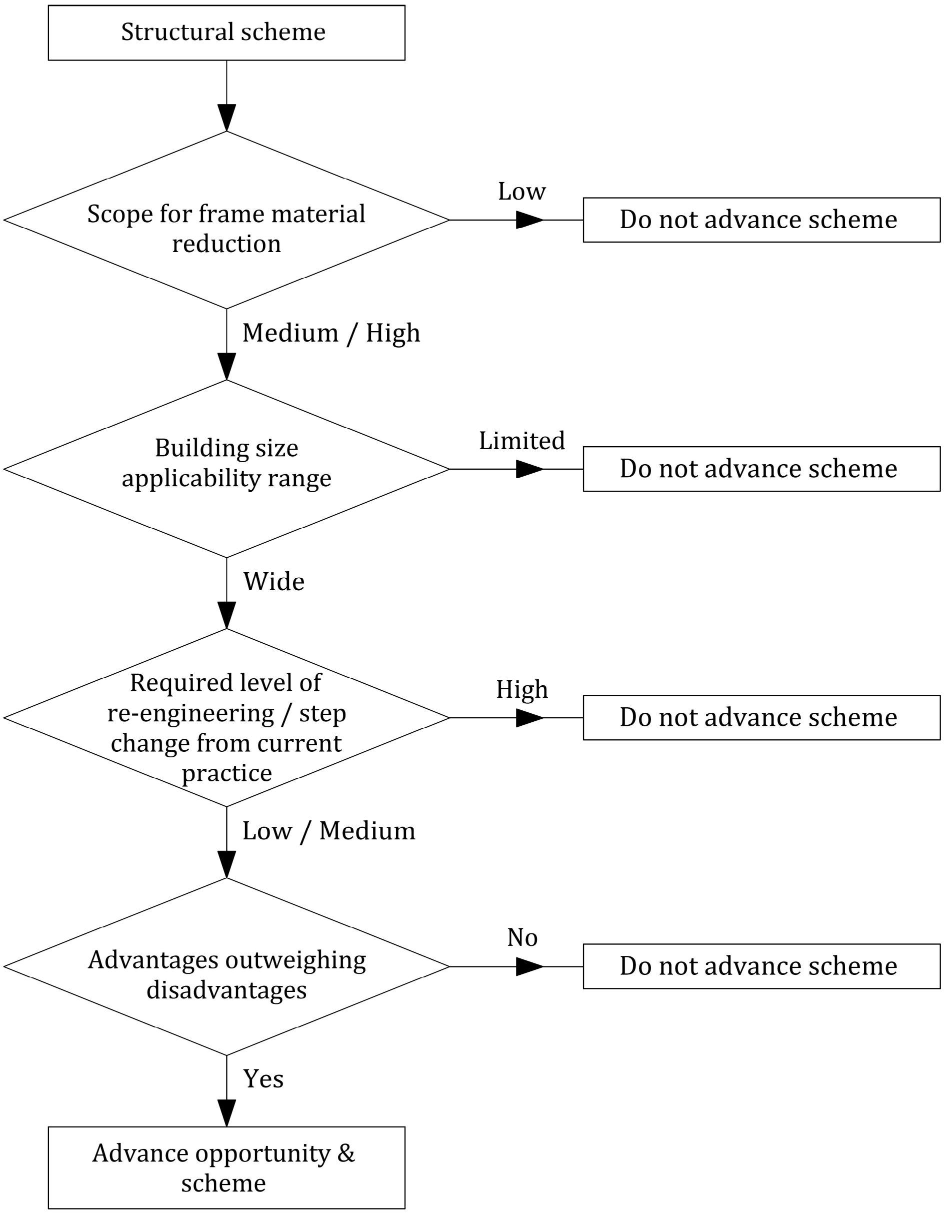


Figure 3.2 Decision-making flow chart

Chapter 4 Buildings with long span roof envelope systems

The current chapter presents the studies undertaken to investigate the feasibility of engineering structure-envelope assemblies which exploit the enhanced structural capability of the envelope in terms of long span roofs. It also presents the structural appraisal of the proposed schemes.

4.1 Building sizes

The building geometries considered in the present study are presented in the Table 4.1.

Table 4.1 Building sizes for long span study

Building size	Width	Length	Height to eaves	Area
Small	25m	40m	4m	1,000m^2
Medium	50m	80m	6m	4,000m^2
Large	80m	125m	6m	10,000m^2

4.2 Structural frame schemes

Four structural frame options were chosen to be examined for the long span opportunity:

1) Duo-pitch portal frames with purlins (base case).
2) Duo-pitch portal frames without purlins.
3) Flat-pitch multi-bay re-oriented portal frames.
4) Frames with trussed roof system and northlights.

The first scheme represents the current practice in the UK for single storey industrial buildings. The other three schemes represent re-engineered structural options chosen to favour long span roof envelope systems. For each scheme option and building size (see Table 4.1), a range of frame spacing distances was applied to assess the effect of varying the number of frames. Each scheme and its associated assumptions and technology barriers are described in the following sections.

The frame spacing cases for the study were chosen between 6.25m and 13.34m to give an integer number of frames for the chosen building geometries and structural schemes. These are summarized later in this chapter. The typical frame spacing in the UK is 6m to 8m.

4.2.1 Duo-pitch portal frames with purlins (base case)

This option represents the current practice in the UK. It comprises duo-pitched portal frames, either single- or multi- bay depending on the building size and clear span requirements. Purlins span between the rafters and the roof cladding spans between the purlins, in a direction from ridge to eaves. This spanning orientation particularly suits profiled roof panels which allow the rainwater to flow in the same direction as the profiles (spanning direction of the cladding). A roof pitch of 6^{o} was chosen, being representative of the UK practice and the tendency for low pitch roofs (Koschmidder and Brown, 2013). A general impression of the structural framing configuration for the portal frames with purlins scheme is shown in Figure 4.1.

In order to achieve the full plastic capacity of the rafter sections by preventing lateral torsional bucking, interaction with the secondary steelwork is required and achieved by the purlins being installed at close spacing, typically 1.8m. The cold-formed steel purlins stabilise the outer flange of the rafters against lateral torsional buckling while the cladding rails perform the same function for the columns. Furthermore, in large bending moment locations (close to the eaves and apex), it is also necessary to restrain the inner flange of the rafter. This is commonly achieved through the use of 'stays'. A similar situation arises at the top of the columns. In addition, the cladding provides restraint to the purlins. The purlins are also used to stabilise the frame during erection against out-of-plane movement.

The lateral stability of the building is provided by the in-plane stiffness of the portal frames in the portal span (in-plane) direction (x-axis) and a bracing system in the transverse (out-of-plane) direction (y-axis). A combination of roof and vertical wall bracing is installed to transfer the wind load acting on the gable façade to the ground. Horizontal eaves struts run longitudinally at the eaves level and across the building edges in order to transfer horizontal forces arising from the lateral load.

The portal frame geometries used for this particular scheme are summarised in Table 4.2.

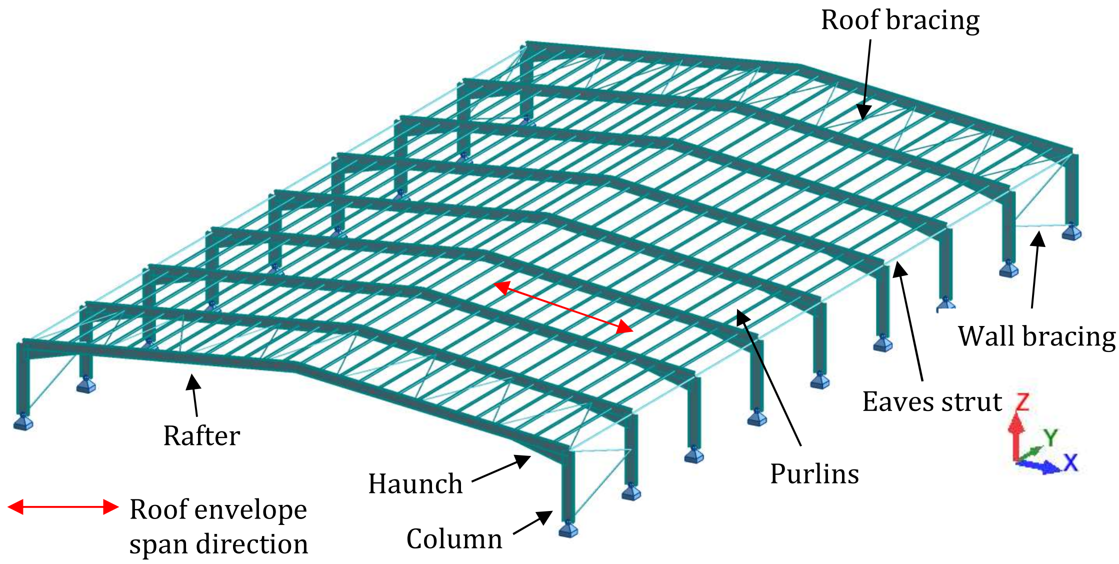


Figure 4.1 Impression of structural scheme of duo-pitch portal frames with purlins

Table 4.2 Frame features for duo-pitch portal frames schemes (with and without purlins)

Building size	Portal frame bays	Portal frame span	Height to eaves	Roof pitch	Total height	Building volume
Small	1	25m	4m	6°	5.31m	4,655m³
Medium	1	50m	6m	6°	8.63m	29,240m³
	2	25m	6m	6°	7.31m	26,620m³
Large	2	40m	6m	6°	8.10m	70,510m³

4.2.2 Duo-pitch portal frames without purlins

This alternative construction option is similar to the previous case, the main difference being the elimination of the purlins and with the roof cladding spanning directly between frames. A general impression of the structural framing configuration for the portal frames without purlins scheme is shown in Figure 4.2. Such a scheme favours the use of long span roof systems, but since the cladding system would need to span horizontally between rafters, a profiled panel could not be used for the external surface. However, a profiled sandwich panel may be used as a base supporting a profiled steel sheet running from ridge to eaves. This arrangement would permit rainwater run-off while allowing the sandwich panels to span between rafters.

The absence of purlins creates the need for an alternative mechanism to restrain the rafters. This may be achieved by the stabilising effect of the roof cladding attached to the top flange of the rafters. This practice is allowed by BS EN 1993-1-1:2005 through the exploitation of the envelope's in-plane shear stiffness. Restraint of the bottom flange may be achieved by installing hollow section members acting as struts between rafters and utilising their axial capacity. This practice may also eliminate the need for temporary frame stabilisation during erection.

The roof / wall bracing and eaves struts systems are identical to those described for the case of the portal frame with purlins.

The portal frame geometries used for this particular scheme are identical to those in Table 4.2.

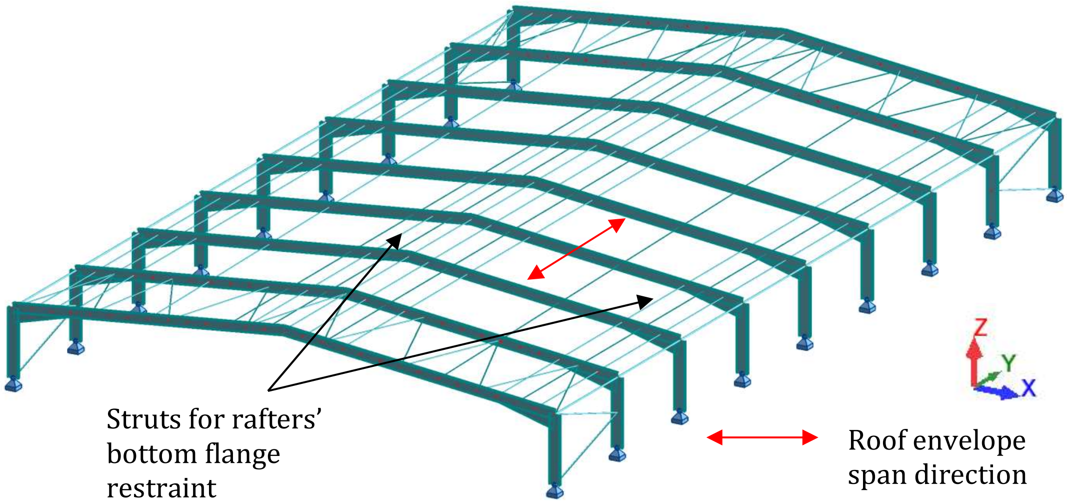


Figure 4.2 Impression of structural scheme of portal frames without purlins

4.2.3 Flat-pitch multi-bay re-oriented portal frames

This structural scheme comprises a series of multi-bay flat roof portal frames spanning in the long direction of the building. A roof pitch of 6^{o} was chosen to keep the same building volume as in previous schemes. Different frame heights are used to match height at the eaves, ridge and intermediate levels of the roof pitch. Re-orienting the frames by 90^{o} allows long span roof cladding to span between frames in a direction from ridge to eaves. This permits rainwater run-off so the profiled sandwich panel may be used as the external cladding surface in this case. An impression of the structural framing configuration for the re-oriented portal frames scheme is shown in Figure 4.3.

Due to the absence of purlins, an alternative means of restraining the rafter is required. As with Option 2, the cladding could be used to restrain the top flange, while the bottom flange is restrained by struts and ties.

The lateral stability of the building is provided by the in-plane stiffness of the portal frames in the portal span (in-plane) direction (y-axis) and a bracing system in the transverse (out-of-plane) direction (x-axis). Horizontal eaves struts run from eaves to ridge at the building edges in order to transfer horizontal forces arising from the lateral load.

The portal frame geometries used for this particular scheme are summarised in Table 4.3.

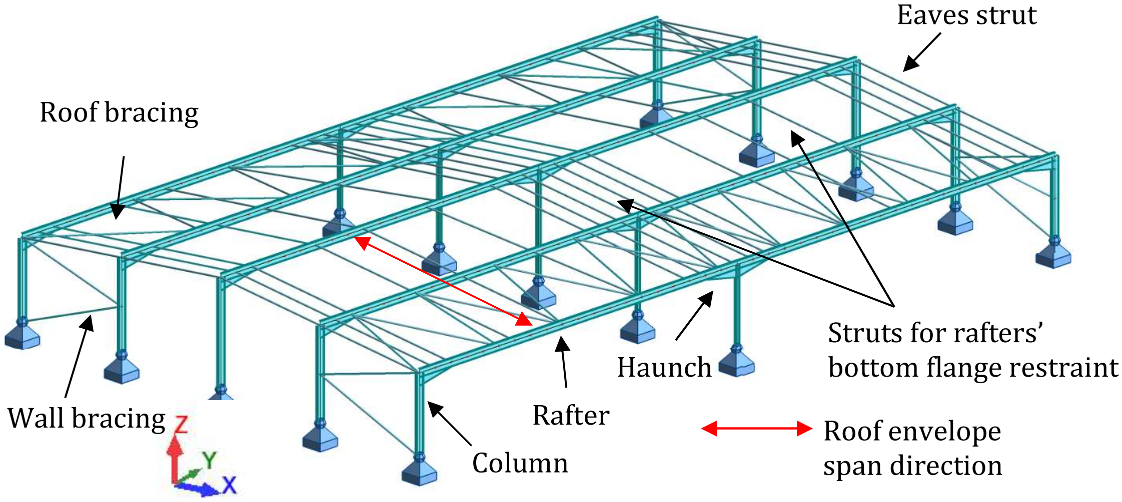


Figure 4.3 Impression of structural scheme with re-oriented portal frames

Table 4.3 Frame features for flat-pitch multi-bay re-oriented portal frames scheme

Building size	Portal frame bays	Portal frame span	Frame height	Frame pitch	Roof pitch	Total height	Building volume
Small	2	20.0m	4m to 5.31m	Flat	6º	5.31m	4,655m³
Medium	4	20.0m	6m to 8.63m	Flat	6º	8.63m	29,240m³
Large	6	20.8m	6m to 9.36m	Flat	6º	8.10m	81,020m³

4.2.4 Trussed roof frames with northlights

This scheme comprises flat roof truss systems which provide clear spans across the width of the building or between widely spaced columns if the building size is large. The northlights are formed by a series of parallel mono-pitch roofs with vertical windows installed within the depth of the truss. The depth of the truss was chosen equal to the eaves to ridge height of the previous schemes so that the building volume is maintained. A general impression of the structural framing configuration for the trussed roof frames with northlights scheme is shown in Figure 4.4. Reference can also be made to Figure 2.4.

The roof cladding system spans between trusses: the top of each roof slope is supported by the upper chord of the truss and the lower edge of the roof slope is supported by the lower chord of the adjacent truss. This spanning orientation particularly suits profiled roof panels which allow the rainwater to flow in the same direction as the profiles (spanning direction of the cladding). Out-of-plane restraint of the top and bottom chords of each truss is provided by in-plane shear stiffness of the roof cladding system, according to the concept in BS EN 1993-1-1:2005. This is a significant advantage over a conventional trussed roof where the cladding only restrains the top chord.

The lateral stability of the building is provided by the in-plane stiffness of the trussed roof frames in the truss span (in-plane) direction (x-axis) and a bracing system in the transverse (out-of-plane) direction (y-axis). The roof and wall bracing systems are identical to the ones described for the duo-pitched portal frames schemes. However, the eaves struts are replaced by transverse trusses across the longitudinal edges of the building acting under tensile and compressive loads.

The trussed roof frame geometries used for this particular scheme are summarised in Table 4.4.

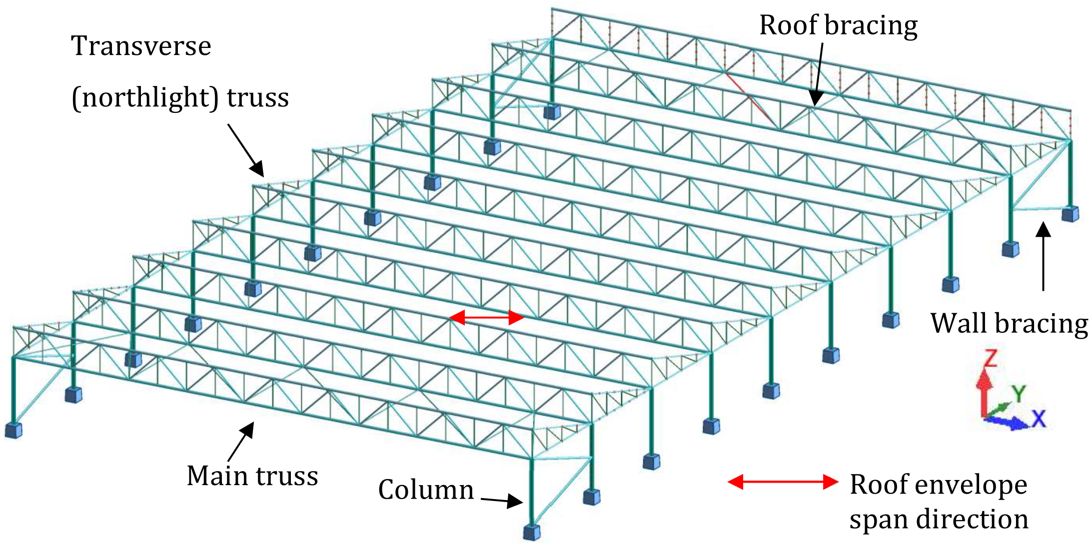


Figure 4.4 Impression of structural scheme of trussed roof frames with northlights

Table 4.4 Frame features for trussed roof frames with northlights scheme

Building size	**Truss bays**	**Truss span**	**Height to eaves**	**Truss height**	**Total height**	**Roof slope**	**Building volume**
Small	1	25m	4m	1.31m	5.31m	7°-11°	4,655m³
Medium	1 and 2	50m	6m	2.63m	8.63m	15°-22°	29,240m³
Large	2	40m	6m	2.10m	8.10m	12°-17°	81,020m³

4.2.5 Frame spacing

The frame spacing cases were chosen so that integer numbers of frames within the building occurred for every building size and structural scheme. The lowest spacing distance of 6.67m represents a typical value used in practice. The spacing cases and number of frames are summarised in Table 4.5. A reference letter was assigned to each spacing distance to group similar magnitudes and facilitate their comparison later.

Table 4.5 Frame spacing cases and number of frames in the building

Building size	Structural scheme	Frame spacing	Spacing reference	No. of frames within building
Small	Duo-pitch portal frames with purlins / Duo-pitch portal frames without purlins / Trussed roof frames with northlights	6.67m	A	7
		8.00m	B	6
		10.00m	C	5
		13.34m	D	4
	Flat-pitch multi-bay re-oriented portal frames	6.25m	A	5
		12.5m	D	3
Medium	Duo-pitch portal frames with purlins / Duo-pitch portal frames without purlins / Trussed roof frames with northlights	6.67m	A	13
		8.00m	B	11
		10.00m	C	9
		13.34m	D	7
	Flat-pitch multi-bay re-oriented portal frames	6.25m	A	9
		8.33m	B	7
		12.5m	D	5
Large	Duo-pitch portal frames with purlins / Duo-pitch portal frames without purlins / Trussed roof frames with northlights	6.67m	A	20
		8.33m	B	16
		10.42m	C	13
		12.50m	D	11
	Flat-pitch multi-bay re-oriented portal frames	6.67m	A	13
		8.88m	B	11
		10.00m	C	9
		13.34m	D	7

4.3 Structural modelling and design

An extensive series of elastic structural analyses were undertaken according to the structural Eurocodes (BS EN 1993-1-1:2005) and the relevant UK National Annex, using the complementary guidance of Koschmidder and Brown (2013), Brown (2013) and Salter *et al*. (2004). Deflection limits for the portal frames were calculated according to SCI (2010). S355 steel grade was used, as is typical in the UK. Struts acting as lateral restraints were designed to resist 1% of the maximum value of the factored force in the compression flange or truss member within the relevant span according to BS 5950-1:2000. Eaves struts were designed to resist horizontal forces equal to 0.5% of the factored load applied on the roof according to BS 5950-1:2000.

A comprehensive list of the modelling input and assumptions is given in Appendix C.

The capability of the envelope to span at the required distances and to provide restraint to the rafters against out-of-plane buckling was not assessed at this stage. This is discussed later in Section 4.5.

4.4 Results and structural appraisal

A summary of the structural steelwork weights per floor area for each of the structural schemes and building sizes is shown in Figure 4.5 to Figure 4.7. The charts have been derived by the information contained in Table 4.6 to Table 4.8 All the quantities include the weight of the frames, purlins, wind bracings, struts and trusses (if present).

A summary of the specified section sizes is given in Appendix C. An analytical breakdown of the steelwork weights per member type for each structural scheme and building type is shown in Table 4.6 to Table 4.8. The relative significance of each member type within each frame's total weight is illustrated in Appendix C.

A summary of the steelwork weight reduction for the various schemes against the base case, together with the optimal frame spacing is shown in Table 4.9.

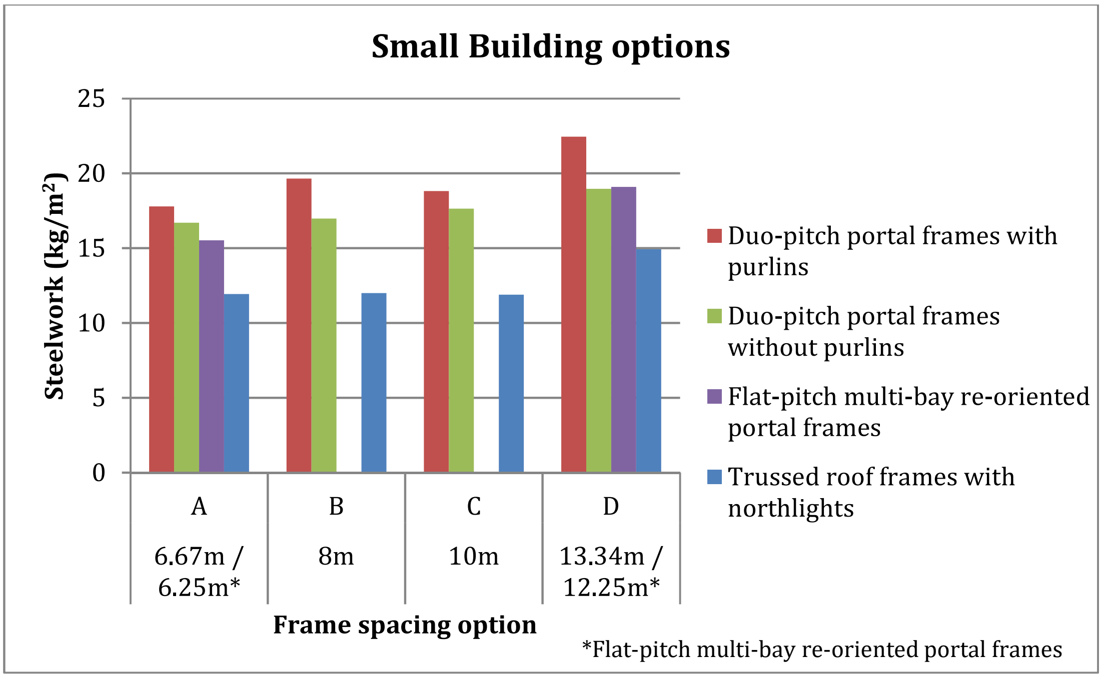


Figure 4.5 Summary of steelwork weights for small building

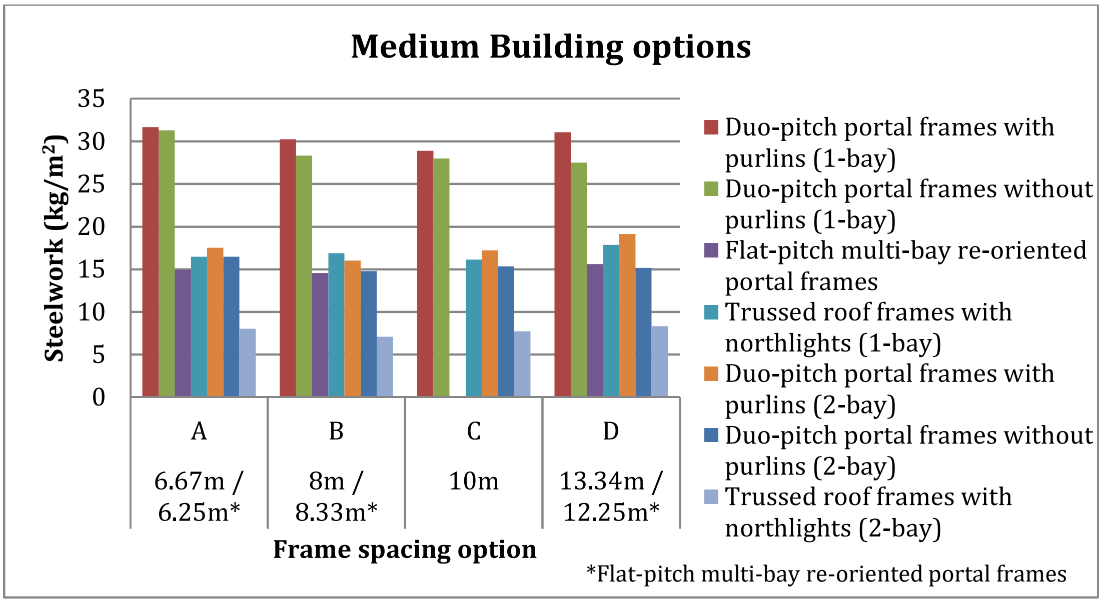


Figure 4.6 Summary of steelwork weights for medium building

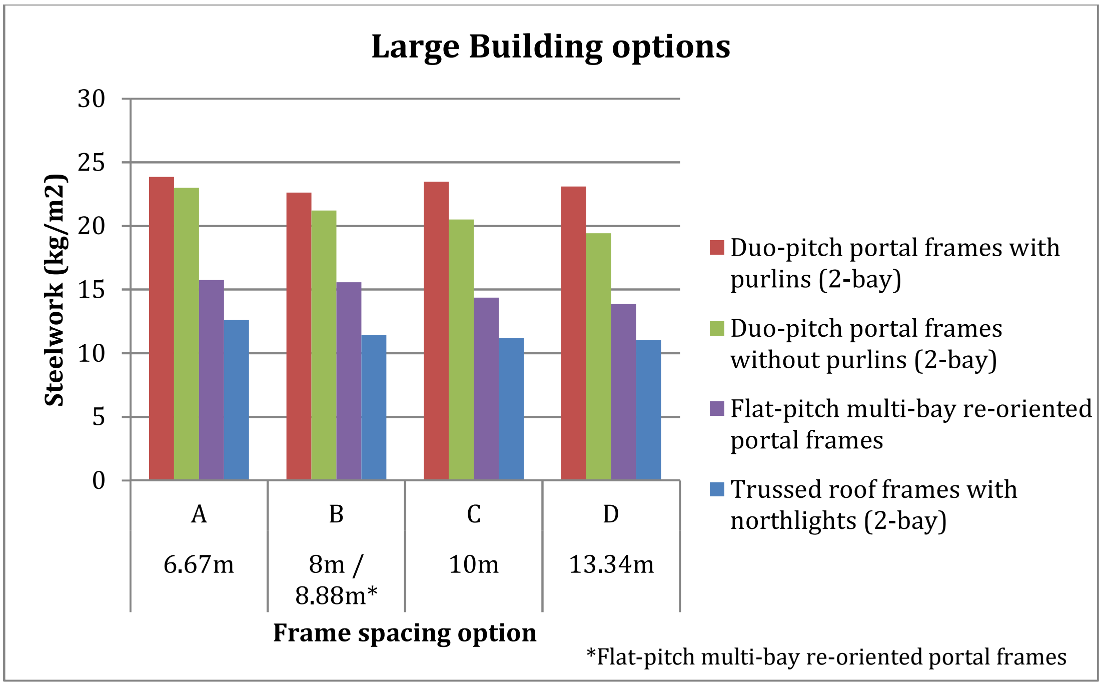


Figure 4.7 Summary of steelwork weights for large building medium building

Table 4.6 Breakdown of steelwork quantities for Small Building

Frame type	Option	Spacing (m)	Frame (tn)	Ties (tn)	Purlins (tn)	Roof bracing (tn)	Wall bracing (tn)	Struts* (tn)	Steel total (tn)	Comparison to	
										Option A	**base case – Option A**
Duo-pitch portal with purlins (1-bay)	A	6.67	13.8		2.1	0.9	0.4	0.5	**17.8**	-	Base Case
	B	8.00	14.7		2.5	1.3	0.6	0.6	**19.6**	+10%	+10%
	C	10.00	12.5		2.9	1.6	1.0	0.9	**18.8**	+6%	+6%
	D	13.34	12.3		5.5	2.4	1.4	0.9	**22.4**	+26%	+26%
Duo-pitch portal without purlins (1-bay)	A	6.67	13.8	1.0		0.9	0.4	0.5	**16.7**	-	-6%
	B	8.00	13.2	1.3		1.3	0.6	0.6	**17.0**	+2%	-5%
	C	10.00	12.5	1.7		1.6	1.0	0.9	**17.6**	+6%	-1%
	D	13.34	12.3	2.0		2.4	1.4	0.9	**19.0**	+14%	+7%
Flat-pitch multi-bay re-oriented portal frames	A	6.25	11.9	1.5		1.3	0.5	0.4	**13.8**	-	-13%
	D	12.25	9.9	3.1		3.5	2.0	0.6	**19.1**	+23%	+7%
Trussed roof frames with northlights (1-bay)	A	6.67	8.8			1.5	0.4	0.2	**11.0**	-	-38%
	B	8.00	8.5			1.8	0.6	0.2	**11.1**	+1%	-38%
	C	10.00	7.6			2.4	0.9	0.2	**11.1**	+1%	-37%
	D	13.34	8.0			4.1	1.8	0.3	**14.2**	+29%	-20%

*Transverse trusses for trussed roof frames

Table 4.7 Breakdown of steelwork quantities for Medium Building

Frame type	Option	Spacing (m)	Frame (tn)	Ties (tn)	Purlins (tn)	Roof bracing (tn)	Wall bracing (tn)	Struts* (tn)	Steel total (tn)	Comparison to	
										Option A	base case – Option A
Duo-pitch portal with purlins (1-bay)	A	6.67	112.8		7.3	2.2	1.1	3.2	**126.6**	-	Base case
	B	8	104.3		8.6	3.0	1.4	3.6	**120.9**	-5%	-5%
	C	10	94.8		11.4	3.7	1.9	3.7	**115.6**	-9%	-9%
	D	13.34	89.0		21.8	6.0	3.2	4.2	**124.2**	-2%	-2%
Duo-pitch portal without purlins (1-bay)	A	6.67	112.8	5.9		2.2	1.1	3.2	**125.1**	-	-1%
	B	8	98.8	6.4		3.0	1.4	3.6	**113.3**	-9%	-10%
	C	10	94.1	8.5		3.7	1.9	3.7	**111.8**	-11%	-12%
	D	13.34	86.7	9.8		6.0	3.2	4.2	**110.0**	-12%	-13%
Duo-pitch portal with purlins (2-bay)	A	6.67	59.1		7.3	1.4	0.6	1.7	**70.1**	-	0%
	B	8	51.3		8.6	1.8	0.7	1.7	**64.1**	-9%	-9%
	C	10	51.2		11.4	2.4	1.1	2.7	**68.9**	-2%	-2%
	D	13.34	45.7		21.8	4.1	1.6	3.2	**76.4**	+9%	+9%
Duo-pitch portal without purlins (2-bay)	A	6.67	59.1	3.0		1.4	0.6	1.7	**65.8**	-	-6%
	B	8	51.3	3.6		1.8	0.7	1.7	**59.1**	-10%	-16%
	C	10	50.5	4.6		2.4	1.1	2.7	**61.3**	-7%	-13%
	D	13.34	45.6	6.1		4.1	1.6	3.2	**60.6**	-8%	-14%
Flat-pitch multi-bay re-oriented portal frames	A	6.25	51.0	2.9		3.3	1.1	1.3	**59.7**	-	-53%
	B	8.33	45.8	4.1		4.9	1.7	1.7	**58.2**	-2%	-54%
	C	12.25	40.9	7.3		8.9	3.0	2.2	**62.3**	+4%	-51%
Trussed roof frames with northlights (1-bay)	A	6.67	58.9			3.5	1.3	0.2	**63.9**	-	-50%
	B	8	54.6			5.1	1.6	0.6	**62.0**	-3%	-51%
	C	10	51.2			6.2	2.0	0.6	**60.1**	-6%	-53%
	D	13.34	54.8			9.5	3.0	0.7	**68.0**	+6%	-46%
Trussed roof frame with northlights (2-bay)	A	6.67	26.8			2.5	0.7	0.2	**30.2**	-	-57%
	B	8	23.5			3.0	1.1	0.1	**27.7**	-8%	-60%
	C	10	23.9			4.6	1.8	0.1	**30.3**	0%	-57%
	D	13.34	25.0			5.4	2.1	0.1	**32.6**	+8%	-53%

*Transverse trusses for trussed roof frames

Table 4.8 Breakdown of steelwork quantities for Large Building

Frame type	Option	Spacing (m)	Frame (tn)	Ties (tn)	Purlins (tn)	Roof bracing (tn)	Wall bracing (tn)	Struts* (tn)	Steel total (tn)	Comparison to	
										Option A	base case – Option A
Duo-pitch portal with purlins (2-bay)	A	6.67	211.2		18.9	2.1	0.6	5.6	**238.5**	-	-
	B	8	190.8		25.5	2.8	0.9	6.0	**226.1**	-5%	-5%
	C	10	175.5		45.2	4.4	1.4	8.3	**234.7**	-2%	-2%
	D	13.34	158.5		55.9	7.1	2.1	7.4	**231.0**	-3%	-3%
Duo-pitch portal without purlins (2-bay)	A	6.67	211.2	10.3		2.1	0.6	5.6	**229.8**	-	-4%
	B	8	189.1	13.1		2.8	0.9	6.0	**211.9**	-8%	-11%
	C	10	171.9	18.9		4.4	1.4	8.3	**204.9**	-11%	-14%
	D	13.34	158.1	19.4		7.1	2.1	7.4	**194.0**	-16%	-19%
Flat-pitch multi-bay re-oriented portal frames	A	6.66	141.9	5.2		6.1	1.4	2.8	**157.5**	-	-34%
	B	8.88	133.5	7.3		9.1	2.2	3.5	**155.5**	-1%	-35%
	C	10	116.4	8.4		11.6	2.8	4.4	**143.5**	-9%	-40%
	D	13.33	100.2	10.9		17.0	4.4	6.1	**138.6**	-12%	-42%
Trussed roof frames with northlights (2-bay)	A	6.66	117.7			3.9	0.8	0.2	**122.6**	-	-49%
	B	8.88	104.7			4.7	1.1	0.2	**110.8**	-10%	-54%
	C	10	99.5			7.2	1.8	0.3	**108.8**	-11%	-54%
	D	13.33	96.2			8.5	2.1	0.3	**107.1**	-13%	-55%

*Transverse trusses for trussed roof frames

Table 4.9 Summary of steelwork reduction, optimum frame spacing, advantages and disadvantages

Scheme	Steelwork reduction (against base case)	Optimum frame spacing	Advantages	Disadvantages	Further work required / Addressing of technical barriers
1 **Duo-pitch portal frame with purlins**	N/A (base case)	Small: 6.67m Medium (1-bay): 10.00m Medium (2-bay): 8.00m Large: 8.33m	• Optimised structural efficiency for the frame. • Established practice. • Purlin orientation suits profiled roof cladding to allow rainwater flow in the same direction as the profiles	• Minimum structural utility of the envelope.	
2 **Duo-pitch portal frame without purlins**	Small: 6% Medium (1-bay): 1%-13% Medium (2-bay): 6%-16% Large: 4%-19%	Small: **6.67m** Medium (1-bay): **8.00m-**13.34m (negligible difference) Medium (2-bay): **8.00m** Large: **10.00m-**13.33m (negligible difference)	• Elimination of purlins and associated cost. • Optimal frame weights may be achieved for small building sizes with the current sandwich panel spanning capability. • Struts may eliminate the need for temporary frame stabilisation during erection.	• Scope for steelwork reduction is relatively small compared to other schemes. • Profiled roof cladding spanning orientation does not allow rainwater to flow, hence cannot be used as external surface. • An additional cladding sheet is required on top of the roof cladding spanning between rafters to permit rainwater run-off.	Engineering of sandwich panels to span 10.00m. Concept-proof of stabilisation of rafters against lateral – torsional buckling.

Continues to next page

				• Sandwich panels with enhanced spanning capabilities (8.00 / 10.00m) are required to achieve the optimal frame weights for medium and large building size. • Detailing for accommodation of rooflights is required to be developed. • Struts are required to provide restraint to the bottom flange of the rafters. • Roof cladding is required to provide restraint and stabilise the top flange of the rafters.	
3 **Flat-pitch multi-bay re-oriented portal frames**	Small: 13% Medium: 51%-53% Large: 34%-42%	Small: **6.25m** Medium: 8.33m / **6.25m** (negligible difference) Large: **10.00m**-13.33m (negligible difference)	• Elimination of purlins and associated cost. • Scope for steelwork reduction is significant for medium and large building sizes compared to other schemes.	• Scope for steelwork reduction is relatively small for small building size compared to other schemes. • Closer clear spans between columns reduce the clear	Engineering of sandwich panels to span 8.00m. Concept-proof of stabilisation of truss chords against lateral-torsional buckling.

			• Profiled roof cladding spanning orientation allows rain water to flow, hence it can be used as external surface. • Optimal frame weights may be achieved for small / medium building sizes with the current sandwich panel spanning capability. • Struts may eliminate the need for temporary frame stabilisation during erection.	areas within the building. • Sandwich panels with enhanced spanning capabilities (10.00m) are required to achieve the optimal frame weights for large building size. • Detailing for accommodation of rooflights is required to be developed. • Struts are required to provide restraint to the bottom flange of the rafters. • Roof cladding is required to provide restraint and stabilise the top flange of the rafters.	
4 **Trussed roof frames with northlights**	Small: 20%-38% Medium (1-bay): 46%-53% Medium (2-bay): 53%-60% Large: 49%-55%	Small: **6.67m** Medium (1-bay): **6.67m**-10.00m (negligible difference) Medium (2-bay): **8.00m** Large: **8.00m**	• Elimination of purlins and transverse trusses and associated cost. • Scope for steelwork reduction is significant for all building sizes	• Intensive fabrication is required for the trusses, compared to portal frames. • North orientation is required. • Sandwich panels with enhanced	Engineering of sandwich panels to span 8.00m.

Continues to next page

			compared to other schemes. • Profiled roof cladding spanning orientation allows rain water to flow, hence it can be used as external surface. • Natural lighting is provided by the northlights without requiring rooflights. • The structural arrangement does not lead to increase of the building volume compared to the other schemes. • Optimal frame weights may be achieved for small / medium building sizes with the current sandwich panel spanning capability. • Long-span panels feasible to provide out-of-plane lateral restraint to truss chords.	spanning capabilities (10.00m) are required to achieve the optimal frame weights for large building size. • Roof cladding is required to provide restraint and stabilise the top and bottom chords of the trusses.	

4.5 Discussion

The following key points are highlighted:

- The steelwork weight for all three re-engineered schemes was lower compared to the base case of duo-pitched portal frames with purlins.
- In terms of minimum steelwork weight, the best performing option was the trussed roof with northlights, followed by the re-oriented portal frames.
- In almost all cases, increasing of the frame spacing resulted in a reduction in steelwork weight. However, the corresponding increase in the weight of the other steel members (purlins, restraining struts, wall and roof bracing systems) offset the reduction in the frame weight in most cases.
- The optimum frame spacing was found to be between 6.25m and 10.00m depending on the building size, with the optimum spacing increasing with building size. Outside this range of spans there was either no reduction in steelwork weight or an insignificant saving.
- In the absence of purlins, the roof envelope system is required to provide adequate stabilisation against the out-of-plane movement of the hot-rolled members (rafters of truss chords). Hollow section strut members were also used to stabilise the bottom flanges of the rafters in the permanent condition, which can as well as provide temporary stabilisation of the frames during erection.
- The trussed roof frames with northlights provided the best option for steelwork reduction (between 38%-60%) while simultaneously offering large clear spans. The optimum frame spacing was found to be 6.67m for the small building size and 8.00m for the medium and large building sizes (the savings for larger distances were negligible). While the building geometries were such that the building volume remains the same as in the other schemes, it is anticipated that a different operational energy performance for the building will apply due to the presence of northlights and the expected variations in lighting and heating. This issue is later discussed in Chapter 8, where the building is holistically reviewed.
- The re-oriented portal frames also appeared to offer an appealing solution in terms of steelwork weight reduction (15%-53%). However, this structural arrangement results in smaller clear spans between columns. The optimum frame spacing was found to be 6.25m for the small and medium building

sizes and 10.00m for the large building size (the savings for larger distances were negligible)

- Duo-pitched portal frames without purlins did not show a very significant steelwork weight reduction (6%-19%) when compared to other schemes. Furthermore, while they would require an extra layer of cladding to allow for rainwater flow. Therefore, steelwork savings through elimination of purlins may be partly outweighed by requirements for additional cladding.

4.6 Structural capability of sandwich panels for long span opportunities

4.6.1 Spanning capabilities

Current roof panel technology offers the capability for long-span applications due to increased insulation depth, as discussed in Section 2.4.4. For a reference roof sandwich panel with modern specifications (full specifications in Appendix A), Table 4.10 shows the increase in spanning capability achieved by the increase of insulation depth alone (i.e. without modification of material and geometrical properties). The results were derived by calculation based on established structural design methodology for sandwich panels (BS EN 14509:2013, Davies *et al*., 2001). The brief analysis shows that modern roof sandwich panels may achieve 6.6m clear span without structural modification (for typical load magnitudes as discussed in Chapter 3 and shown in Appendix B). This would particularly suit the optimum spanning requirements identified for the small building size. For the medium and large buildings, improved sandwich panel spanning capability is required to achieve spans at 8m, corresponding to the optimum frame spacing. In order for sandwich panels to achieve the longer spans, re-engineering of the panels would be required, comprising adjustments to the geometrical and mechanical properties of the different material layers.

Table 4.10 Spanning capability of roof sandwich panels with various insulation thicknesses

Insulation thickness	Maximum span	Available products
120mm	6.1m	Yes
135mm	6.6m	Yes
150mm	7.4m	No
180mm	8.3m	No

In order to realise the optimum long span sandwich panel solutions, further research work is required to specify roof sandwich panels to span at 8m to suit the most favourable frame schemes for medium and large buildings. The panels would be likely to possess higher embodied carbon due to increased material usage. Consequent embodied carbon increases would also need to be quantified. This is investigated further in Chapter 7.

4.6.2 Restraining capabilities

The long-span roof envelope systems would be spanning directly between frames. In the absence of purlins, the envelope would be required to provide restraint to the frame members against lateral-torsional buckling for the portal frame rafters and out-of-plane lateral buckling for the truss chords of the trussed-roof frames with northlights. For the latter, torsion would be prevented due to the adopted hollow sections for the top and bottom truss chords. In order to provide restraint, the roof envelope and its connections would be required to resist in-plane shear forces arising as a result of bending moments in the rafters or truss girders.

The current section investigates the capability of sandwich panels to provide the required restraint to the primary structure. As discussed in Section 4.7, the scheme comprising trussed-roof frames with northlights offers the greatest opportunity for frame material reduction and is the only scheme advanced for further research. Hence, the investigation in the current section is narrowed to sandwich panels providing out-of-plane lateral restraint to the truss chords.

The capability of long span sandwich panel assemblies to provide lateral restraint to the truss chords is demonstrated by conceptual verification according to the provisions in BS 5950-9:1994 for sheeting and decking. The standard allows the cladding to be used as diaphragm bracing and to stabilise rafters or beams. Where fewer than three beams need to be stabilised, the envelope must resist a total force of not less than the sum of the lateral restraint forces required for each beam, being minimum 2.5% of the maximum factored force in the compression flange and distributed uniformly across the length of the beam (BS 5950-9:1994). ECCS-CIB (2013) includes provisions for restraint of beams against lateral buckling with the aid of sandwich panels; however, the manual merely includes assemblies with edge fasteners only and has not matured enough to include seam-fastened panels, which are typical in the UK for weather-tightness purposes.

The diaphragm resistance of the cladding depends on the capacity and spacing of the edge and seam fasteners, the presence of shear connectors at the longitudinal edge of the diaphragm panel and whether the panel is fastened on two sides (to beams only) or four sides with shear connectors (BS 5950-9:1994, Bryan and Davies, 1984). The design guidance in BS 5950-9:1994 is based on the research of Lawson and Nethercot (1985) for the use of sheeting and is applicable for the most common failure modes. For sandwich panels in in-plane shear, failure typically occurs at the fasteners, while profile distortion, shear strain in sheeting and shear buckling are prevented by the stabilising function of the stiff core to the thin sheets as observed in earlier research (Davies and Lawson, 1999). This approach is also adopted in the *'Recommendations for Stabilisation of Steel Structures by Sandwich Panels'* (ECCS-CIB, 2013) where the panels are considered infinitely stiff and their shear resistance and flexibility are governed by the fasteners.

An analysis was undertaken to conceptually examine the ability of long-span sandwich panels to restrain the truss-chord members for the building cases in the present study. The following span distances were selected to match the frame spacing identified as optimum in Section 4.4 and 4.5:

- 6.67m for the small building
- 8.00m for the medium and large buildings

The analysis was undertaken according to BS 5950-9:1994. The diaphragm panel providing restraint is formed by the truss span and the spacing of the frame. The edge and seam fastener capacities for sandwich panels were calculated according to ECCS-CIB (2013), assuming the steel thickness of the reference roof sandwich panel, being 0.5mm for the outer sheet and 0.4mm for the inner sheet (full specifications in Appendix A). Shear connectors were deemed to possess the same shear capacity as the edge fasteners.

The results of the analysis according to BS 5950-9:1994 in terms of stabilising forces and fastener spacing are shown in Table 4.11.

Table 4.11 Sandwich panel out-of-plane lateral restraint to truss chords

Building size	**Small**	**Medium**	**Large**
Diaphragm depth (frame spacing)	6.67m	8.00m	8.00m
Diaphragm length (truss length)	25m	50m	80m
Max in-plane compressive force	532.5kN	290kN	611.4kN
Lateral restraint force requirement	26.6kN	14.5kN	30.6kN
UDL equivalent	1.07kN/m	0.29kN/m	0.38kN/m
Seam fastener spacing (max)	425mm	425mm	425mm
Edge fastener spacing	333mm	333mm	333mm
Shear connectors spacing (max)	445mm	1000mm	475mm

The results show that conventionally fastened sandwich panels spanning directly between trusses at the identified optimum span distances can easily stabilise the truss chord members and provide out-of-plane lateral restraint. This is feasible with a modest increase of fasteners at the diaphragm's edges through the use of shear connectors. For the scheme comprising trussed-roof frames with north lights, this justifies the suitability of seam-fastened sandwich panels spanning between trusses to adequately restrained the truss chord members and achieve efficient design for the structure.

The approach adopted for this conceptual verification was simplified on the basis of BS 5950-9:1994, assuming a uniform distribution of the stabilising force across the chord based on the maximum bending moment in the truss. In reality, the distribution of in-plane forces would be non-uniform. Further investigation of this mode would benefit from further research; however is outside the scope of the present study.

4.7 Decision-making on opportunity advancement

A summary of the scope for structural material reduction, the advantages / disadvantages and the associated barriers associated with the long span opportunity and each scheme is shown in Table 4.9. The results and intermediate steps of the decision-making process are shown in Table 4.12. The decision whether to take each opportunity and scheme forward is made based on the decision-making process described in Chapter 3.

The trussed roof frames with northlights particularly and the re-oriented portal frames were found to yield a significant benefit in terms of material reduction, whilst they can be implemented for the whole range of building sizes. The trussed roof frames with northlights were found to yield benefits which outweighed their disadvantages and addressing the associated technical barriers is considered to be low risk. The re-oriented portal frames, on the other hand, require a compromise in terms clear areas, due to the close column spacing. For modern industrial buildings, the clear space is typically required to be obstructed as little as possible to accommodate various uses which change through time. The re-oriented portal frames would then be imposing a significant compromise on the usability aspect.

Therefore, it was decided that the trussed roof frames with northlights option at optimum frame spacing (6.67m for the small building size and 8.00m for the medium and large building sizes) would be taken forward for further research.

Table 4.12 Decision-making for long span opportunity and schemes

Scheme	Scope for steelwork reduction	Building size applicability range	Technical barriers / requirements for further research	Risk of technical barriers not being addressed	Step change from current practice	Advantages outweighing the disadvantages	Decision on researching the opportunity further
2 **Duo-pitch portal frame without purlins**	Small for all building sizes.	Wide.	Engineering of sandwich panels to span 8.00m to 10.00m. Concept-proof of stabilisation of rafters against lateral-torsional buckling.	Low.	Small.	No.	No.
3 **Flat-pitch multi-bay re-oriented portal frames**	Yes Significant for medium / large building sizes. Small for small building sizes.	Wide.	Engineering of sandwich panels to span 10.00m. Concept-proof of stabilisation of rafters against lateral-torsional buckling.	Low.	Medium.	Yes, if extent of clear space within the building is not an issue.	No.
4 **Trussed roof frames with northlights**	Yes Significant for all building sizes.	Wide.	Engineering of sandwich panels to span 8.00m.	Low.	Small.	Yes.	Yes.

Chapter 5 Buildings with diaphragm action

The current chapter presents the studies undertaken to investigate the feasibility of engineering structure-envelope assemblies which exploit the enhanced structural capability of the envelope in terms of diaphragm action. It also presents the structural appraisal of the proposed schemes.

5.1 Building sizes

The building geometries considered in the present study are presented in the Table 5.1.

Table 5.1 Building sizes for diaphragm action study

Building size	Width	Length	Height to eaves	Area
Small	25m	40m	4m	1,000m^2
Medium	50m	80m	6m	4,000m^2
Large	80m	125m	6m	10,000m^2

5.2 Structural frame schemes

The opportunity to exploit the diaphragm action of the cladding was examined for the following four structural frame schemes:

1. Duo-pitch portal frames with purlins:
    a. with normal (6^o) roof pitch (base case)
    b. with high (12^o) roof pitch
2. Duo-pitch portal frames without purlins and long span roof envelope spanning between rafters:
    a. with normal (6^o) roof pitch
    b. with high (12^o) roof pitch

Scheme 1a represents the current practice in the UK for single storey industrial buildings, while Scheme 1b is common for farm buildings. The other two schemes represent the re-engineered structural options with the greatest potential to exploit diaphragm action.

Diaphragm action typically has a greater effect on buildings with steeper roofs, where the design is dominated by 'sway' failure mode (typically for wind plus snow load combination). However, in the UK, there is a tendency for portal frames to have relatively low pitch roofs, between 5^{o} to 10^{o} and commonly 6^{o} (Koschmidder and Brown, 2013, Brown, 2013), which shifts the dominant failure mode to 'spread' (typically for permanent plus imposed load combination). For this reason, two roof pitches (6^{o} and 12^{o}) were considered.

Furthermore, the shear stiffness of the roof diaphragm depends on the structural arrangement of the roof and particularly on the number and type of components (e.g. purlins, fasteners, shear connectors) and connections. For this reason, two span arrangements (between purlins and between rafters) were considered, suiting the aforementioned frame schemes 1 and 2. For each spanning arrangement the effect of normal and dense end and seam fastening was investigated to examine the limits of the envelope's stiffness and resistance.

The portal frames used in this analysis were identical to those described in Sections 4.1.1 and 4.1.2. The portal frame geometries were the same as in Table 4.2 for the normal pitch case. The higher pitch case was of similar geometry but with a 12^{o} roof pitch and a corresponding increase in total height and building volume as shown in Table 5.2. A general impression of the structural framing configuration with high roof pitch is shown in Figure 5.1 for the portal frames with purlins scheme and Figure 5.2 for the portal frames without purlins scheme.

A typical frame spacing of approximately 6.6m was chosen for each building size, to give an integer number of frames.

Table 5.2 Frame features for duo-pitch portal frames schemes with high roof pitch (with and without purlins)

Building size	No. of bays	Portal span	Height to eaves	Roof pitch	Total height	Frame spacing	Building volume
Small	1	25m	4m	12^{o}	6.66m	6.67m	5,328m^3
Medium	1	50m	6m	12^{o}	11.31m	6.67m	34,628m^3
	2	25m	6m	12^{o}	9.15m	6.67m	34,628m^3
Large	2	40m	6m	12^{o}	10.25m	6.58m	81,256m^3

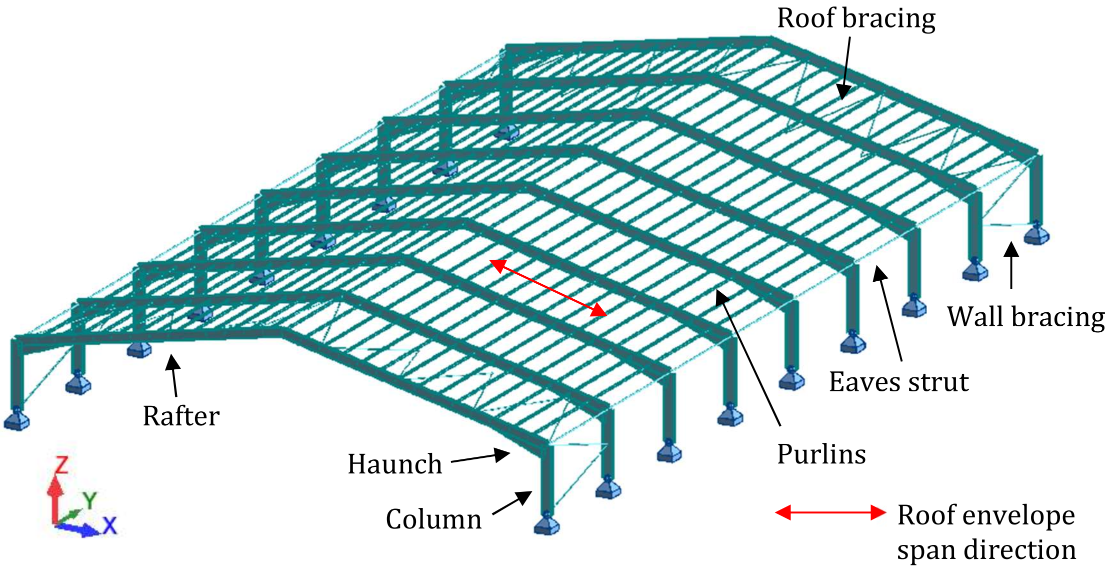


Figure 5.1 Impression of structural scheme of duo-pitch portal frames with purlins and high roof pitch

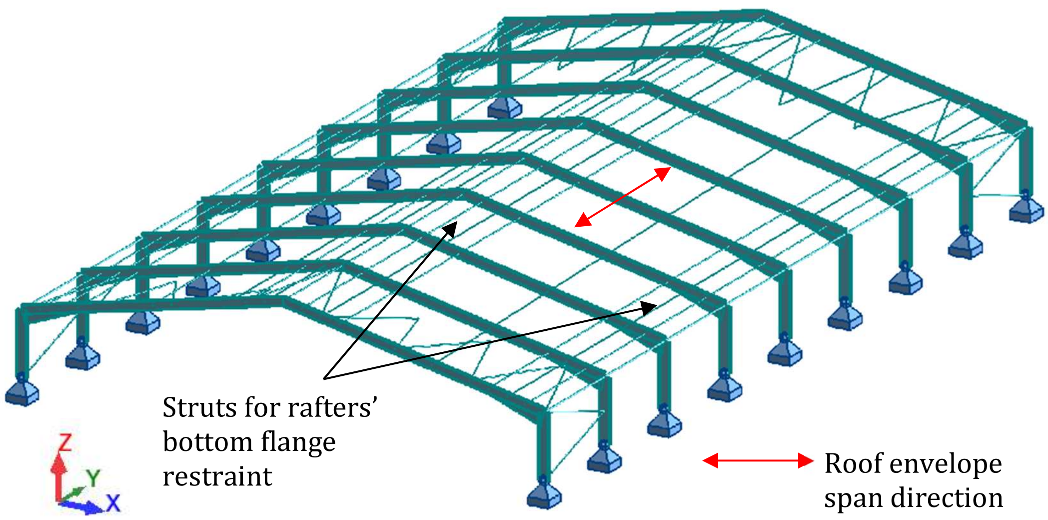


Figure 5.2 Impression of structural scheme of duo-pitch portal frames without purlins and high roof pitch

In theory, roof diaphragm action also offers the potential to eliminate roof bracing. However, the results of the structural appraisal presented in Table 4.6, Table 4.7 and Table 4.8 of the earlier chapter showed that the roof bracing is only a small percentage of the total steelwork in the building. A calculation shows that roof bracing weights correspond to 5.1%, 2.0% and 0.8% of the total steelwork in the building for the small, medium and large buildings respectively. Furthermore, the roof bracing typically stabilises the frame during erection and provides a stiff framework for the installation of purlins and cladding. Therefore, due to the limited scope for benefit and the potential frame erection issues, the option of eliminating the roof bracing with the aid of diaphragm action was not included in the present study.

The study took no account of the presence of rooflights since this represents the best case for diaphragm action exploitation. According to BS 5950-9:1994, effects on diaphragm action from openings in the roof of less than 3% of the roof area may be disregarded. However, modern industrial buildings typically possess opening at approximately 12% of the roof area for natural lighting and energy conservation purposes. Therefore, if the diaphragm action opportunity was to be advanced, further studies on the effects and provisions for openings would be required.

5.3 Structural modelling and design

A series of modern sandwich panel arrangements were modelled based on the principles of BS 5950-9:1994 and their in-plane shear flexibilities and resistances were calculated.

For the purposes of the feasibility study, the reference roof sandwich panel system with modern specifications was assumed with the following geometry:

- 0.5mm external steel (S220) face with 32mm deep profiles and stiffened troughs
- 0.4mm internal (liner) lightly profiled steel (S220) face
- 135mm PIR core (ρ=38kg/m^3) to cover likely future thermal requirements for notional building (U=0.15W/m^{2}K)

The full panel specification is given in Appendix A.

The fastening resistances and flexibilities for the sandwich panel assemblies were estimated according to Käpplein and Ummenhofer (2011). Measured rather than nominal data for the panel sheets of the reference panel were used (see Appendix A) in order to estimate connection strengths and flexibilities as close to practice as possible. The panels were modelled to span either between purlins or rafters, to suit Schemes 1 and 2 respectively (see Figure 5.3). A shear panel is outlined by the ridge and eaves purlins or beams and two adjacent rafters.

The core was considered to be infinitely stiff compared to the connections and also prevent profile distortion and shear buckling of the sheets. This approach is consistent with the design guidance in Käpplein and Misiek (2011g), as well as the findings of tests on sandwich panel diaphragms (Davies and Lawson, 1999, Mahendran and Subaaharan, 2002).

Different fastening arrangements were investigated including combinations of: (a) normal and dense end and seam fastening and (b) fastening on two or four sides of the shear panels. The case of 2-sided fastened panels with normal end and seam fastener spacing corresponds to a typical cladding installation without diaphragm action provisions. 4-sided arrangements and dense fastening correspond to structurally enhanced cases. A sketch is shown in Figure 5.3.

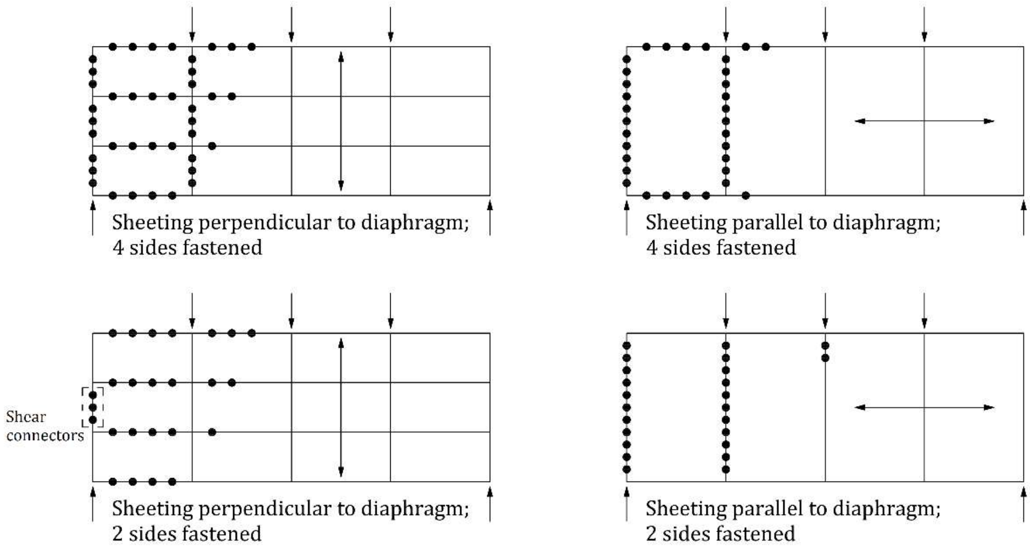


Figure 5.3 Panel span and fastener arrangements (adapted from BS 5950-9:1994)

The portal frames were modelled according to the principles of BS EN 1993-1-1:2005, using a procedure as described in Section 4.3. The coupled behaviour of the shear diaphragms and the rigid-jointed portal frames was modelled according to the provisions of BS 5950-9:1994. This is based on the concept that the cladding will provide an alternative load path so that a distribution of loading between frame and cladding occurs, leading to a reduction of the moments and forces developed in the frames. The extent of the diaphragm action depends on the in-plane shear flexibility of the cladding relative to the frame flexibility. The stiffer the cladding relative to the frame, the greater the diaphragm action stiffening effect.

In order to take into account the stiffening effect of the cladding, a reduction may be applied to the 'sway' and 'spread' bending moment of the bare frame, based on the number of frames within the building and the analytical model given by BS 5950-9:1994. Scope for re-design of the frames is offered by the code by allowing for modification of forces in the frames due to the diaphragm action effect. The analysis followed the procedure in Section 7 in BS 5950-9:1994, which provides a simple methodology similar to the one that would normally be used by a small or medium design consultancy without the need for specialist software. An important requirement of Section 7 is that that all frames within a given building were identical and the gable frames were fully braced. More discussion is made in Section 5.4.4.

5.4 Results and structural appraisal

5.4.1 Shear panel flexibilities and resistances

The results of the shear panel flexibilities and resistances of the various sandwich panel arrangements (spanning between purlins or rafters, normal or dense fastening) are shown in Table 5.3 (normal seam fastener spacing) and Table 5.4 (dense seam fastener spacing). The lower the shear panel flexibility, the higher its stiffness and the potential for diaphragm action contribution. The detailed modelling input is included in Appendix D. The shear resistances and flexibilities of the fastening arrangements used in the study are included in Table D.1. The governing failure mode which dominates the shear resistance was found to be bearing of the steel sheets for both end and seam fasteners.

As the results in Table 5.3 and Table 5.4 indicate, fastening the panels on 4 sides was the best means of achieving meaningful diaphragm action, together with the spanning arrangement between rafters for Scheme 2. The use of a dense end fastening regime provided only a small increase in envelope stiffness compared to normal fastening conditions. The increase of seam fastener density on the other hand led to considerable increase of the stiffness and resistance.

Table 5.3 Resistances and flexibilities of sandwich shear panel arrangements (normal seam fastener spacing)

Shear panel dimensions – ref. building	**Spanning between**	**Resistance and flexibility**			
		Normal end fastening		Dense end fastening	
		4-sides	2-sides	4-sides	2-sides
12.6m x 6.67m (Small, 1-bay; Medium 2-bay)	Purlins (Scheme 1)	24.3kN * 0.083mm/kN	18.3kN ** 0.529mm/kN	27.3kN * 0.075mm/kN	25.6kN ** 0.494mm/kN
	Rafters (Scheme 2)	45.6kN * 0.053mm/kN	45.6kN * 0.493mm/kN	47.2kN * 0.052mm/kN	47.2kN * 0.478mm/kN
25.2m x 6.67m (Medium, 1-bay)	Purlins (Scheme 1)	48.2kN * 0.036mm/kN	48.2kN * 0.274mm/kN	53.7kN * 0.034mm/kN	53.7kN * 0.257mm/kN
	Rafters (Scheme 2)	91.1kN * 0.025mm/kN	91.1kN * 0.138mm/kN	94.4kN * 0.024mm/kN	94.4kN * 0.133mm/kN
20.1m x 6.58m (Large, 2-bay)	Purlins (Scheme 1)	38.0kN * 0.048mm/kN	38.0kN * 0.345mm/kN	42.4kN * 0.044mm/kN	42.4kN * 0.323mm/kN
	Rafters (Scheme 2)	73.3kN * 0.031mm/kN	73.3kN * 0.204mm/kN	76.3kN * 0.031mm/kN	76.3kN * 0.197mm/kN

*Seam failure; **Failure of end fasteners at internal rafter

Table 5.4 Resistances and flexibilities of sandwich shear panel arrangements (dense seam fastener spacing)

Shear panel dimensions – ref. building	Spanning between	Resistance and flexibility			
		Normal end fastening		Dense end fastening	
		4-sides	2-sides	4-sides	2-sides
12.6m x 6.67m (Small, 1-bay; Medium 2-bay)	Purlins (Scheme 1)	35.9kN *** 0.049mm/kN	18.3kN ** 0.495mm/kN	55.3kN * 0.043mm/kN	25.6kN ** 0.462mm/kN
	Rafters (Scheme 2)	92.5kN * 0.034mm/kN	92.5kN * 0.474mm/kN	94.1kN * 0.034mm/kN	94.1kN * 0.460mm/kN
25.2m x 6.67m (Medium, 1-bay)	Purlins (Scheme 1)	104.3kN * 0.019mm/kN	75.6kN * 0.257mm/kN	109.8kN * 0.018mm/kN	105.8kN * 0.241mm/kN
	Rafters (Scheme 2)	184.9kN * 0.015mm/kN	184.9kN * 0.128mm/kN	188.2kN * 0.015mm/kN	188.2kN * 0.123mm/kN
20.1m x 6.58m (Large, 2-bay)	Purlins (Scheme 1)	82.0kN * 0.026mm/kN	82.0kN * 0.323mm/kN	86.4kN * 0.023mm/kN	86.4kN * 0.302mm/kN
	Rafters (Scheme 2)	149.6kN * 0.019mm/kN	149.6kN * 0.191mm/kN	152.2kN * 0.019mm/kN	152.2kN * 0.185mm/kN

*Seam failure; **Failure of end fasteners at internal rafter; ***Panel / purlin fastener capacity

The sandwich panels were also benchmarked against built-up and decking systems, showing that they are an ideal means of diaphragm action in terms of stiffness, although of lower strength when compared to decking. The stiffness benefit is primarily related to the in-plane stiffness and stabilising effect of the core, which minimise the effect of modes such as profile distortion and shear strain in the steel sheets. The lower strength is primarily due to the thinner gauge of the sandwich panel liner, resulting in bearing failure of the fasteners. The detailed results of the benchmarking study are shown in Table D.2 in Appendix D and a direct comparison to Table 5.3 can be made.

5.4.2 Relative flexibilities and load transfer

The sway and spread frame flexibilities (k_{sw} and k_{sp}, as defined in Figure 5.4) were calculated and are shown in Table 5.5. The relative flexibilities of the shear panels (shown in in Table 5.3 and Table 5.4) over the flexibilities of the portal frames for both sway and spread modes (r_{sw} and r_{sp}, shown in Table 5.5) were calculated and shown in Table 5.6 to Table 5.9 for all the building sizes and number of bays. For the 4-sided arrangements, a normal end and a dense seam fastener arrangement were chosen to represent a reasonable upper bound of the response. Dense end fastening was avoided since it showed little improvement in terms of strength and stiffness. Also, due to implications such as denser repeating thermal bridging and further reduction of cross-sectional panel resistance at the intermediate supports of continuous panel arrangements under wind uplift (BS EN 14509:2013). For the 2-sided arrangement, normal end and seam fastener arrangement were chosen to represent the as-built conditions and the lower bound of the response. A low relative flexibility (cladding / frame) value shows a high cladding stiffness relative to the frame, hence a higher contribution of diaphragm action. For example, a comparison of the relative sway flexibility values for 4-sided (0.030) versus 2-sided (0.322) fastened panels in Scheme 1a for a small building size (see Table 5.6) indicates that the cladding stiffness is more significant relative to the frame for the 4-sided fastened case and, therefore, the contribution of diaphragm action is greater.

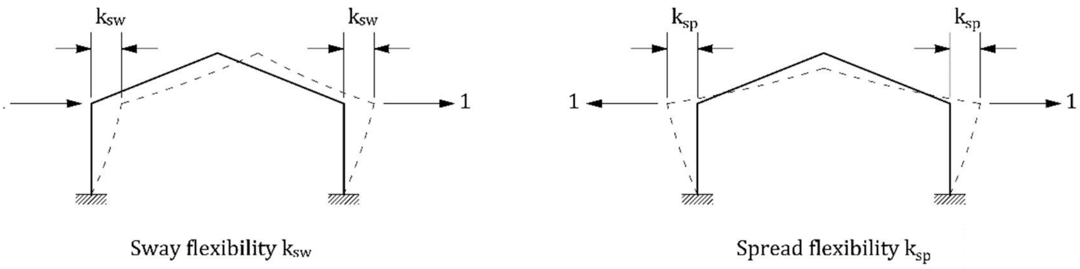


Figure 5.4 Definition of sway and spread frame flexibilities k_{sw} and k_{sp} (adapted from BS 5950-9:1994)

Table 5.5 Frame flexibilities

Building	**No. of bays**	**Frame pitch**	**Sway flexibility k_{sw} (mm/kN)**	**Spread flexibility k_{sp} (mm/kN)**
Small	1	6°	1.644	0.040
		12°	1.744	0.116
Medium	1	6°	1.261	0.046
		12°	1.442	0.138
Medium	2	6°	3.320	0.068
		12°	3.182	0.203
Large	2	6°	1.606	0.081
		12°	1.505	0.219

Table 5.6 Relative flexibilities of sandwich panel arrangements and frames (Small building size, 1-bay frame)

Spanning between	**Frame pitch**	**Mode**	**Relative flexibility r_{sw} & r_{sp} (cladding / frame)**		**Frame failure mode**
Purlins (Scheme 1)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.030	0.322	Spread
		Spread (r_{sp})	1.225	13.225	
	High (12°) (Scheme b)	Sway (r_{sw})	0.028	0.303	Spread + Sway
		Spread (r_{sp})	0.422	4.560	
Rafters (Scheme 2)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.021	0.300	Spread
		Spread (r_{sp})	0.850	12.325	
	High (12°) (Scheme b)	Sway (r_{sw})	0.019	0.283	Spread + Sway
		Spread (r_{sp})	0.293	4.250	

[*]Normal end fastener density, dense seam fastener density; [**]Normal end and seam fastener density

Table 5.7 Relative flexibilities of sandwich panel arrangements and frames (Medium building size, 1-bay frame)

Spanning between	Frame pitch	Mode	Relative flexibility r_{sw} & r_{sp} (cladding / frame)		Frame failure mode
Purlins (Scheme 1)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.015	0.217	Spread
		Spread (r_{sp})	0.413	5.957	
	High (12°) (Scheme b)	Sway (r_{sw})	0.013	0.190	Spread + Sway
		Spread (r_{sp})	0.138	1.986	
Rafters (Scheme 2)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.012	0.109	Spread
		Spread (r_{sp})	0.326	3.000	
	High (12°) (Scheme b)	Sway (r_{sw})	0.010	0.096	Spread + Sway
		Spread (r_{sp})	0.109	1.000	

**Normal end fastener density, dense seam fastener density; **Normal end and seam fastener density*

Table 5.8 Relative flexibilities of sandwich panel arrangements and frames (Medium building size, 2-bay frame)

Spanning between	Frame pitch	Mode	Relative flexibility r_{sw} & r_{sp} (cladding / frame)		Frame failure mode
Purlins (Scheme 1)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.015	0.159	Spread
		Spread (r_{sp})	0.721	7.779	
	High (12°) (Scheme b)	Sway (r_{sw})	0.015	0.166	Spread + Sway
		Spread (r_{sp})	0.241	2.606	
Rafters (Scheme 2)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.010	0.148	Spread
		Spread (r_{sp})	0.500	2.429	
	High (12°) (Scheme b)	Sway (r_{sw})	0.011	0.155	Spread + Sway
		Spread (r_{sp})	0.167	2.429	

**Normal end fastener density, dense seam fastener density; **Normal end and seam fastener density*

Table 5.9 Relative flexibilities of sandwich panel arrangements and frames (Large building size, 2-bay frame)

Spanning between	Frame pitch	Mode	Relative flexibility r_{sw} & r_{sp} (cladding / frame)		Frame failure mode
Purlins (Scheme 1)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.016	0.215	Spread
		Spread (r_{sp})	0.321	4.259	
	High (12°) (Scheme b)	Sway (r_{sw})	0.017	0.229	Spread + Sway
		Spread (r_{sp})	0.119	1.575	
Rafters (Scheme 2)	Normal (6°) (Scheme a)	Sway (r_{sw})	0.012	0.127	Spread
		Spread (r_{sp})	0.235	2.519	
	High (12°) (Scheme b)	Sway (r_{sw})	0.013	0.136	Spread + Sway
		Spread (r_{sp})	0.087	0.932	

**Normal end fastener density, dense seam fastener density; **Normal end and seam fastener density*

Based on the relative flexibility ratios, the ratio of the load distributed to the bare frame is shown in Table 5.10 to Table 5.13, while the remaining load is applied to the envelope. These ratios correspond to the reduction factors η_{sw} and η_{sp} which may be applied to the frame according to the calculation method in BS 5950-9:1994. The application of the method requires that all frames within the building are similar (Clause 7.1 of BS 5950-

9:1994). The reduction factors indicate the amount of load carried by the frame when diaphragm action is exploited as a percentage of the load which would be carried by the bare frame without diaphragm action. As an example, for 4-sided fastened panels spanning between purlins in Scheme 1a for a small building size (see Table 5.10), the load ratio of 0.648 for the penultimate frames in a spread mode indicates that when the contribution of diaphragm action is taken into account, the given frame is required to resist only 64.8% of the load which would be resisted without diaphragm action. Therefore, low values of the load ratio shown in Table 5.10 to Table 5.13 indicate a high opportunity to reduce the frame size.

Table 5.10 Ratio of applied load resisted by the bare frame and shear panel resistances for selected arrangements (Small building, 1-bay frames)

<table>
<tr><th rowspan="3">Shear panel</th><th rowspan="3">No. of frames</th><th rowspan="3">Frame location</th><th colspan="4">Load ratio resisted by frame</th><th rowspan="3">Shear panel resistance (kN)</th></tr>
<tr><th colspan="2">Normal roof pitch (6°) (Scheme a)</th><th colspan="2">High roof pitch (12°) (Scheme b)</th></tr>
<tr><th>η_{sway}</th><th>η_{spread}</th><th>η_{sway}</th><th>η_{spread}</th></tr>
<tr><td rowspan="3">2-sided fastening, spanning between purlins* (Scheme 1)</td><td rowspan="3">7</td><td>2, 6 (Penultimate)</td><td>0.389</td><td>0.934</td><td>0.377</td><td>0.844</td><td rowspan="3">18.3</td></tr>
<tr><td>3, 5</td><td>0.582</td><td>0.996</td><td>0.566</td><td>0.975</td></tr>
<tr><td>4 (Intermediate)</td><td>0.640</td><td>0.999</td><td>0.623</td><td>0.992</td></tr>
<tr><td rowspan="3">4-sided fastening, spanning between purlins** (Scheme 1)</td><td rowspan="3">7</td><td>2, 6 (Penultimate)</td><td>0.068</td><td>0.648</td><td>0.064</td><td>0.443</td><td rowspan="3">35.9</td></tr>
<tr><td>3, 5</td><td>0.108</td><td>0.865</td><td>0.101</td><td>0.651</td></tr>
<tr><td>4 (Intermediate)</td><td>0.121</td><td>0.916</td><td>0.114</td><td>0.712</td></tr>
<tr><td rowspan="3">4-sided fastening, spanning between rafters** (Scheme 2)</td><td rowspan="3">7</td><td>2, 6 (Penultimate)</td><td>0.049</td><td>0.581</td><td>0.045</td><td>0.371</td><td rowspan="3">92.5</td></tr>
<tr><td>3, 5</td><td>0.078</td><td>0.805</td><td>0.071</td><td>0.557</td></tr>
<tr><td>4 (Intermediate)</td><td>0.087</td><td>0.863</td><td>0.080</td><td>0.614</td></tr>
</table>

**Normal end and seam fastener density; **Normal end fastener density, dense seam fastener density*

Table 5.11 Ratio of applied load resisted by the bare frame and shear panel resistances for selected arrangements (Medium building, 1-bay frames)

Shear panel	No. of frames	Frame location	Load ratio resisted by frame: Normal roof pitch (6°) (Scheme a)		Load ratio resisted by frame: High roof pitch (12°) (Scheme b)		Shear panel resistance (kN)
			η_{sway}	η_{spread}	η_{sway}	η_{spread}	
2-sided fastening, spanning between purlins* (Scheme 1)	13	2, 12 (Penultimate)	0.366	0.872	0.346	0.731	48.2
		3, 11	0.595	0.984	0.568	0.928	
		4, 10	0.735	0.998	0.708	0.981	
		5, 9	0.818	1.000	0.792	0.995	
		6, 8	0.862	1.000	0.837	0.998	
		7 (Intermediate)	0.875	1.000	0.852	0.999	
4-sided fastening, spanning between purlins** (Scheme 1)	13	2, 12 (Penultimate)	0.069	0.468	0.061	0.300	104.3
		3, 11	0.125	0.716	0.110	0.503	
		4, 10	0.167	0.847	0.148	0.638	
		5, 9	0.196	0.914	0.175	0.723	
		6, 8	0.214	0.946	0.190	0.770	
		7 (Intermediate)	0.220	0.955	0.195	0.785	
4-sided fastening, spanning between rafters** (Scheme 2)	13	2, 12 (Penultimate)	0.057	0.429	0.049	0.267	184.9
		3, 11	0.103	0.673	0.088	0.455	
		4, 10	0.138	0.810	0.118	0.583	
		5, 9	0.163	0.884	0.139	0.666	
		6, 8	0.178	0.921	0.152	0.712	
		7 (Intermediate)	0.183	0.932	0.156	0.727	

**Normal end and seam fastener density; **Normal end fastener density, dense seam fastener density*

Table 5.12 Ratio of applied load resisted by the bare frame and shear panel resistances for selected arrangements (Medium building, 2-bay frames)

Shear panel	**No. of frames**	**Frame location**	**Load ratio resisted by frame**				**Shear panel resistance (kN)**
			Normal roof pitch (6°) (Scheme a)		High roof pitch (12°) (Scheme b)		
			η_{sway}	η_{spread}	η_{sway}	η_{spread}	
2-sided fastening, spanning between purlins* (Scheme 1)	13	2, 12 (Penultimate)	0.320	0.897	0.326	0.772	18.3
		3, 11	0.532	0.989	0.541	0.948	
		4, 10	0.670	0.999	0.679	0.988	
		5, 9	0.755	1.000	0.764	0.997	
		6, 8	0.801	1.000	0.810	0.999	
		7 (Intermediate)	0.816	1.000	0.825	1.000	
4-sided fastening, spanning between purlins** (Scheme 1)	13	2, 12 (Penultimate)	0.069	0.562	0.069	0.382	35.9
		3, 11	0.125	0.808	0.125	0.615	
		4, 10	0.167	0.915	0.167	0.756	
		5, 9	0.196	0.962	0.196	0.837	
		6, 8	0.214	0.981	0.214	0.879	
		7 (Intermediate)	0.220	0.986	0.220	0.892	
4-sided fastening, spanning between rafters** (Scheme 2)	13	2, 12 (Penultimate)	0.049	0.500	0.053	0.327	92.5
		3, 11	0.088	0.749	0.096	0.542	
		4, 10	0.118	0.873	0.128	0.681	
		5, 9	0.139	0.934	0.151	0.766	
		6, 8	0.152	0.961	0.165	0.812	
		7 (Intermediate)	0.156	0.969	0.170	0.826	

**Normal end and seam fastener density; **Normal end fastener density, dense seam fastener density*

Table 5.13 Ratio of applied load resisted by the bare frame and shear panel resistances for selected arrangements (Large building, 2-bay frames)

Shear panel description	No. of frames	Frame location	Load ratio resisted by frame				Shear panel resistance (kN)
			Normal roof pitch (6°) (Scheme a)		High roof pitch (12°) (Scheme b)		
			η_{sway}	η_{spread}	η_{sway}	η_{spread}	
2-sided fastening, spanning between purlins* (Scheme 1)	20	2, 19 (Penultimate)	0.368	0.836	0.377	0.694	38.0
		3, 18	0.601	0.973	0.612	0.906	
		4, 17	0.748	0.996	0.758	0.971	
		5, 16	0.840	0.999	0.849	0.991	
		6, 15	0.898	1.000	0.905	0.997	
		7, 14	0.934	1.000	0.940	0.999	
		8, 13	0.956	1.000	0.960	1.000	
		9, 12	0.968	1.000	0.972	1.000	
		10, 11 (Intermediate)	0.974	1.000	0.977	1.000	
4-sided fastening, spanning between purlins** (Scheme 1)	20	2, 19 (Penultimate)	0.098	0.428	0.102	0.290	82.0
		3, 18	0.181	0.673	0.189	0.495	
		4, 17	0.251	0.813	0.261	0.639	
		5, 16	0.309	0.893	0.322	0.741	
		6, 15	0.356	0.939	0.370	0.812	
		7, 14	0.393	0.964	0.408	0.861	
		8, 13	0.420	0.979	0.436	0.893	
		9, 12	0.438	0.986	0.455	0.913	
		10, 11 (Intermediate)	0.447	0.990	0.464	0.922	
4-sided fastening, spanning between rafters** (Scheme 2)	20	2, 19 (Penultimate)	0.079	0.381	0.084	0.252	149.6
		3, 18	0.148	0.617	0.156	0.440	
		4, 17	0.206	0.763	0.218	0.578	
		5, 16	0.254	0.853	0.269	0.680	
		6, 15	0.294	0.908	0.311	0.755	
		7, 14	0.325	0.942	0.343	0.807	
		8, 13	0.348	0.962	0.367	0.843	
		9, 12	0.363	0.973	0.383	0.866	
		10, 11 (Intermediate)	0.371	0.979	0.391	0.877	

**Normal end and seam fastener density; **Normal end fastener density, dense seam fastener density*

A comparison between Table 5.6 to Table 5.9 and Table 5.10 to Table 5.13 shows that the lower the relative flexibility (hence the higher the cladding stiffness relative to the frame's), the lower the force which is transmitted to the bare frame and, consequently, the higher the force transmitted to the envelope. In other words, the lower the relative flexibility, the greater the contribution of diaphragm action. The reduction in bare frame forces was more significant for the 'sway' mode compared to the 'spread' mode due to the relatively high shear stiffness of the envelope compared to the frame. As an example, the results for a 4-sided fastened panel in Scheme 1a for a small building size (Table 5.6) show that the relative flexibility in a sway mode (0.030) is much smaller compared to the value in a spread mode (1.225). Consequently, any frame in the building (e.g. the penultimate ones) would need to resist much lower loads under sway (6.8% of the frame load without diaphragm action) compared to spread (64.8% of the frame load without diaphragm action) (see Table 5.10).

Section 7 in BS 5950-9:1994 allows the derived load factors to be used in elastic analysis to recalculate forces and deflections, provided that all frames within the building are similar. In theory, the re-calculations of the forces distributed between frame and envelope raises the potential of re-designing the frame to account for reduced forces and deflection. A reduction of section sizes due to reduced frame loading is, however, not automatically guaranteed. Frame member sizes are chosen from a set of standard cross-sections and corresponding resistances. In general, a considerable reduction in frame loading is required in order for a section size drop to be applicable. For the cases shown in Table 5.6 to Table 5.9, it is unlikely that resizing of members occurs if the load ratio is higher than 0.9, i.e. if the load distributed to the bare frame is maintained at 90% and above. There are also other reasons behind limitations in frame re-design despite the apparent load frame reduction. These are discussed in more detail in Section 5.4.4.

5.4.3 Frame deflections and forces in the envelope

The derived reduction factors from Table 5.10 to Table 5.13 and the consequential modification of the forces in frames and envelope may be used in elastic analysis to recalculate the deflections and forces for the selected sandwich panel arrangements, i.e. the as-currently-installed case and the enhanced solutions for the purlin and long span arrangements.

The earlier calculated shear panel resistances may be used:

- To assess the level of load required to be resisted by the envelope due to its relative stiffness compared to the frame and the consequential activation of diaphragm action.
- For frame design accounting for the envelope reaching its plastic resistance and sustaining large plastic deformations (more discussion in made in Section 5.4.4).

Figure 5.5, Figure 5.8, Figure 5.11 and Figure 5.14 show the reduction of eaves deflections compared to those of the bare frames by using the load factors shown in Table 5.10 to Table 5.13 and the deflections originally calculated for the bare frame without diaphragm action. The deflections were calculated for ‘wind only’ load case at SLS, which is critical for sway deflections.

Figure 5.6, Figure 5.9, Figure 5.12 and Figure 5.15 show the relevant apex case. The deflections were calculated for 'wind plus snow' load case at SLS, which is critical for spread deflections.

Figure 5.7, Figure 5.10, Figure 5.13 and Figure 5.16 show that the level of force developed in the envelope at SLS (wind only load case) as a percentage of the envelope's resistance. This is if no roof bracing is present at the end gables. Practically, the end roof panels (i.e. the panel between frames 1 and 2) will be braced, hence the shown exceedance of shear panel resistance would not be critical at these locations.

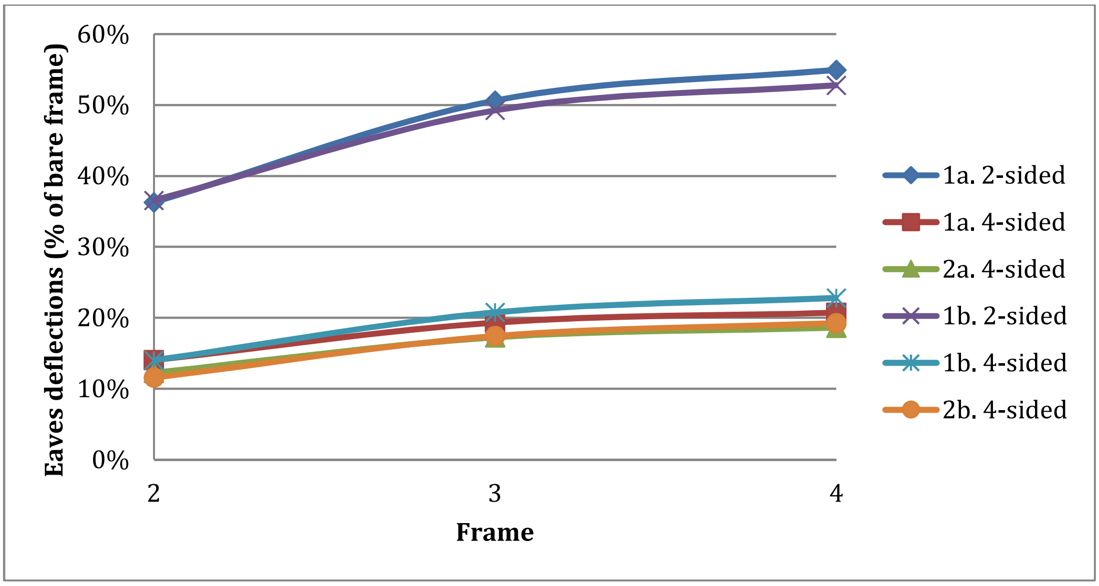


Figure 5.5 Eaves deflections (Small building size, 1-bay)

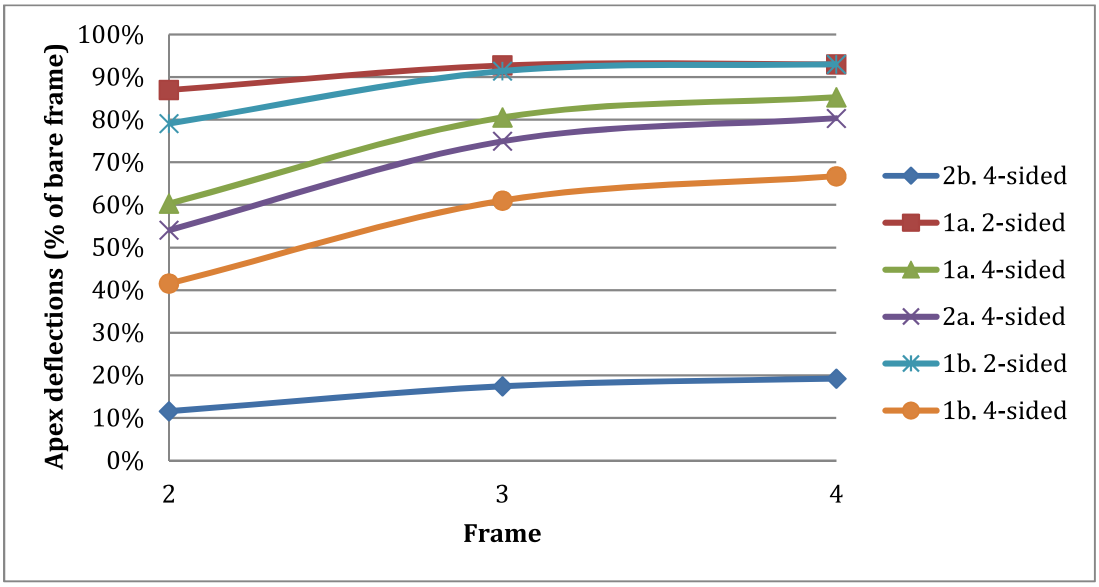


Figure 5.6 Apex deflections (Small building size, 1-bay)

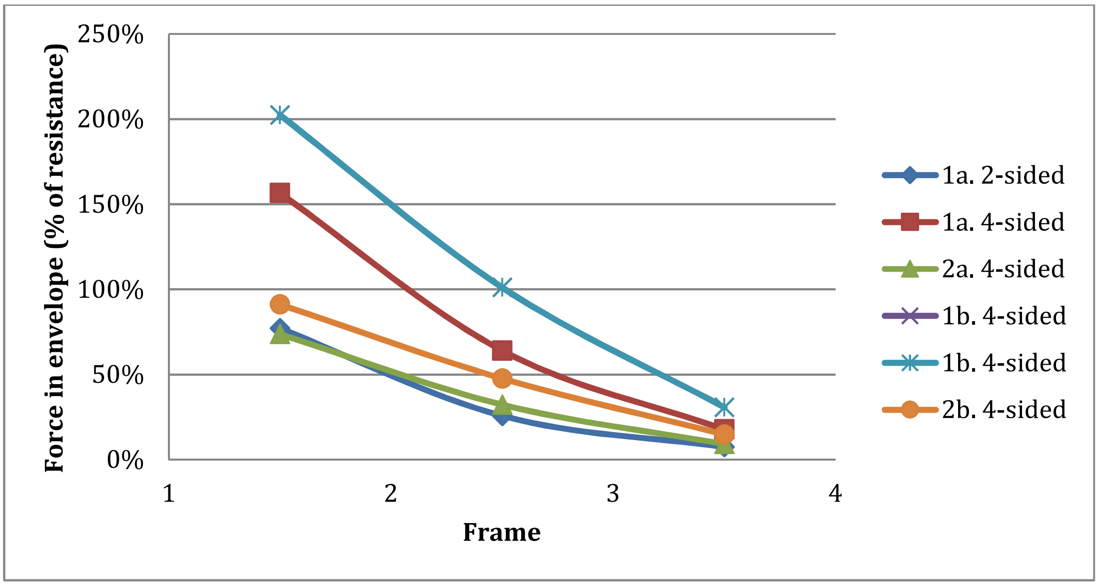


Figure 5.7 Force in the envelope – SLS (Small building size, 1-bay)

Note: End bay not normally critical due to presence of roof bracing.

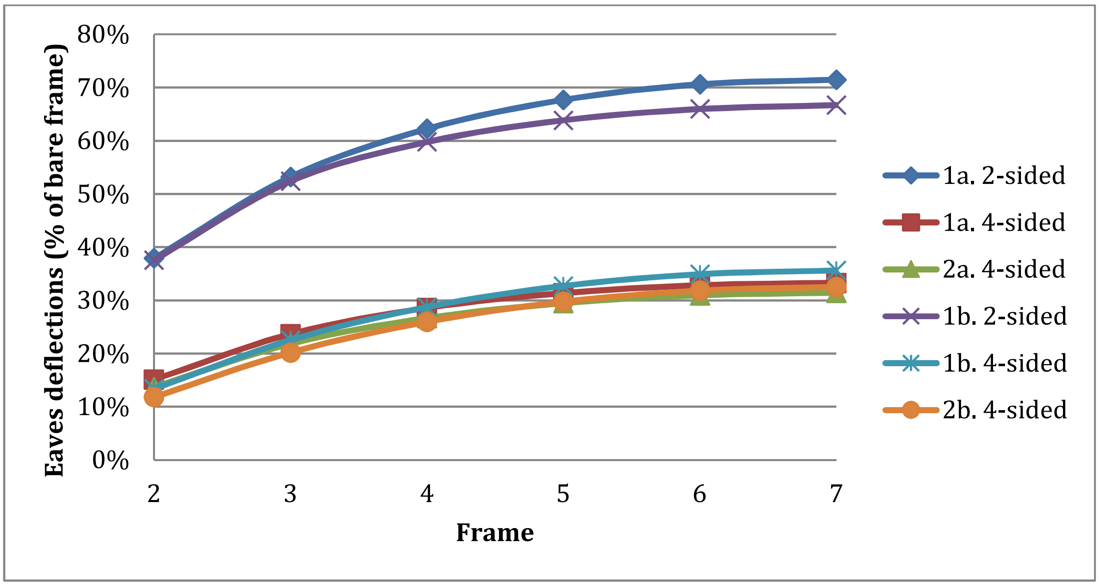


Figure 5.8 Eaves deflections (Medium building size, 1-bay)

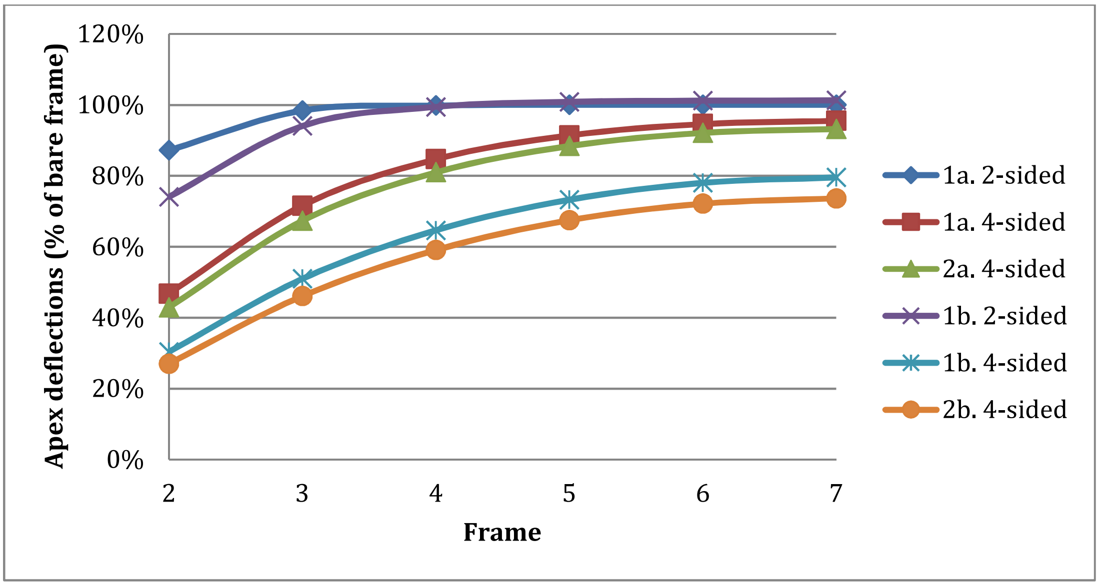


Figure 5.9 Apex deflections (Medium building size, 1-bay)

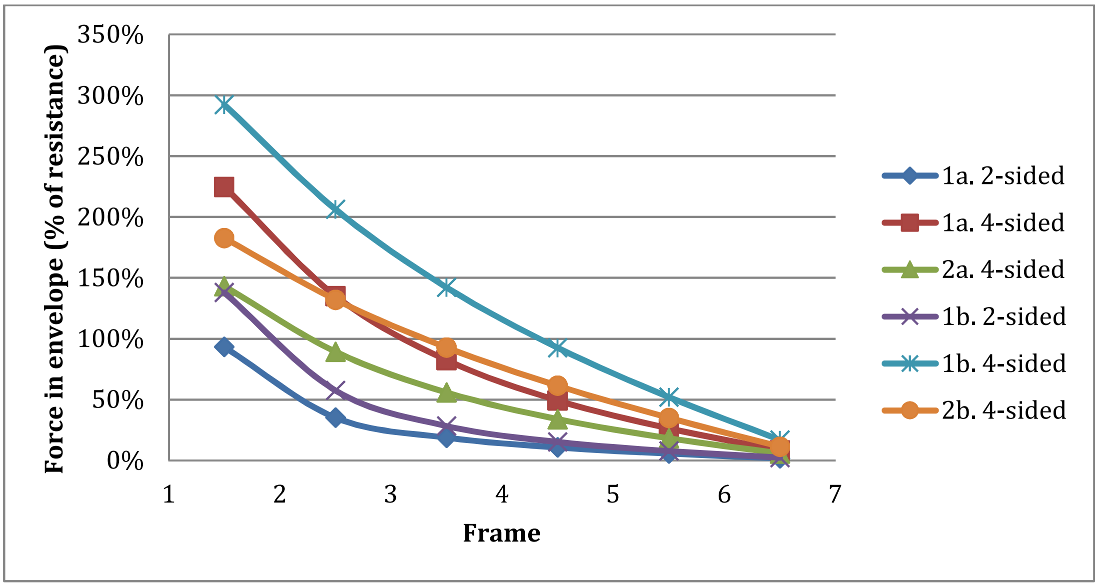


Figure 5.10 Force in the envelope – SLS (Medium building size, 1-bay)

Note: End bay not normally critical due to presence of roof bracing.

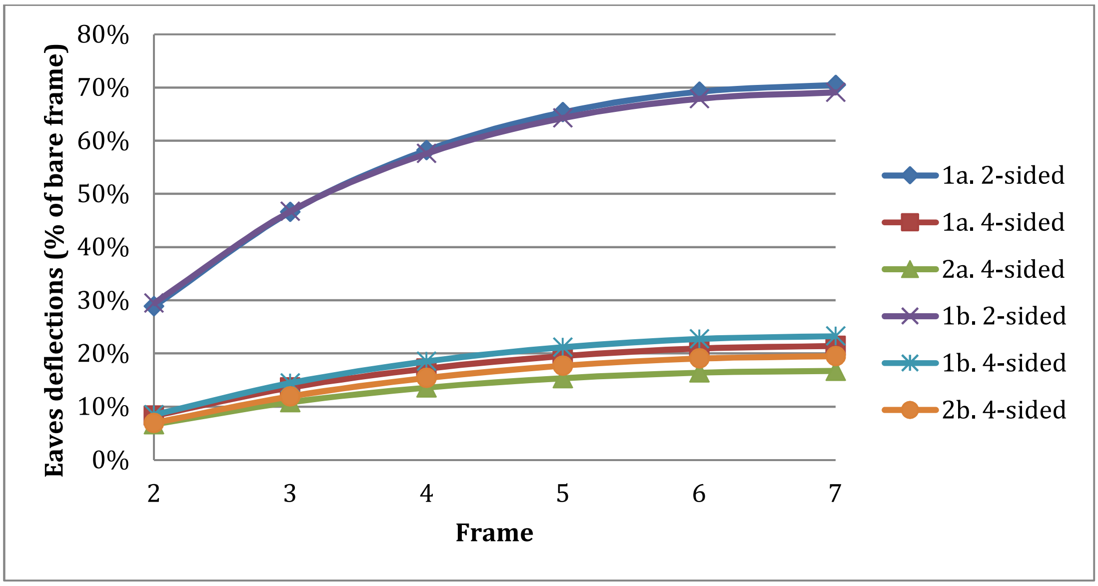


Figure 5.11 Eaves deflections (Medium building size, 2-bay)

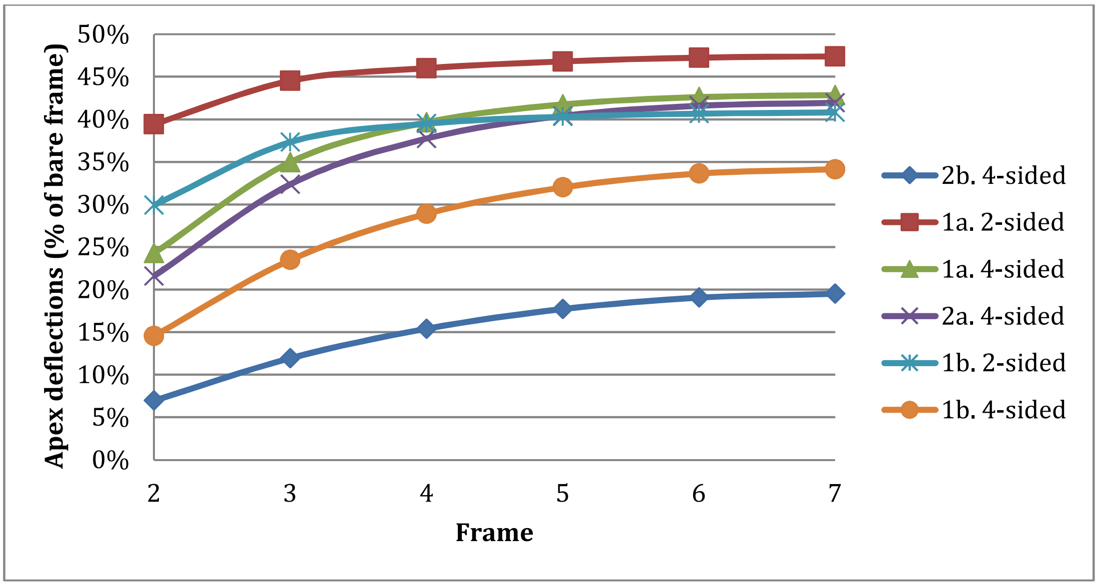


Figure 5.12 Apex deflections (Medium building size, 2-bay)

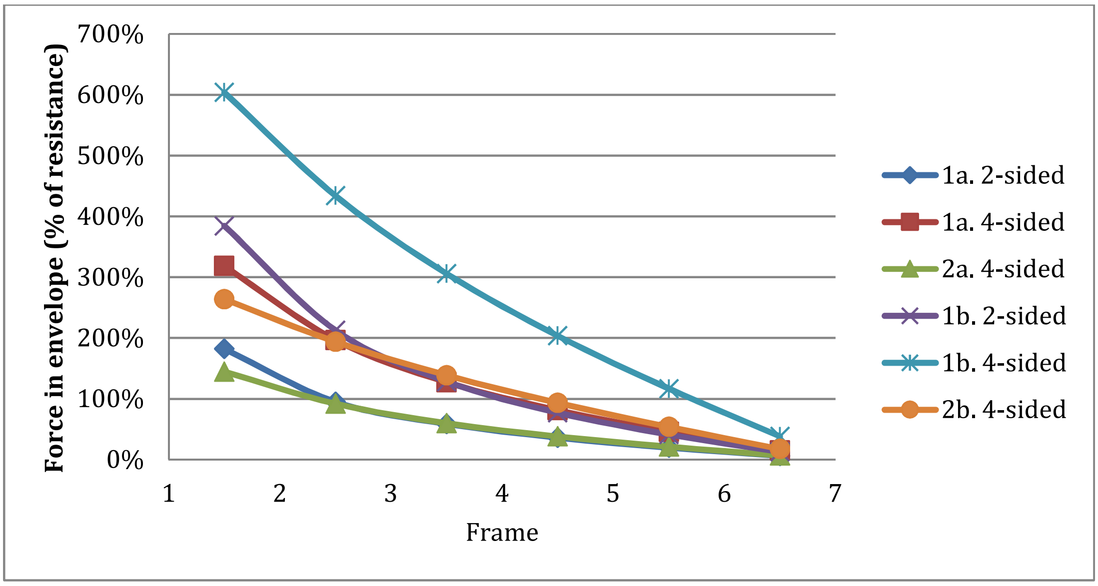


Figure 5.13 Force in the envelope – SLS (Medium building size, 2-bay)

Note: End bay not normally critical due to presence of roof bracing.

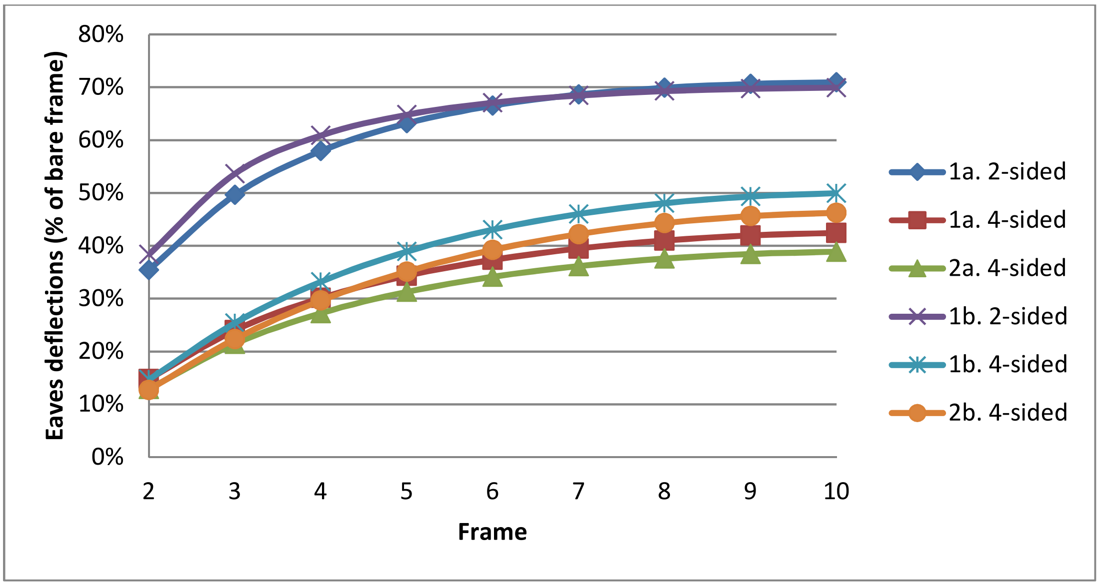


Figure 5.14 Eaves deflections (Large building size, 2-bay)

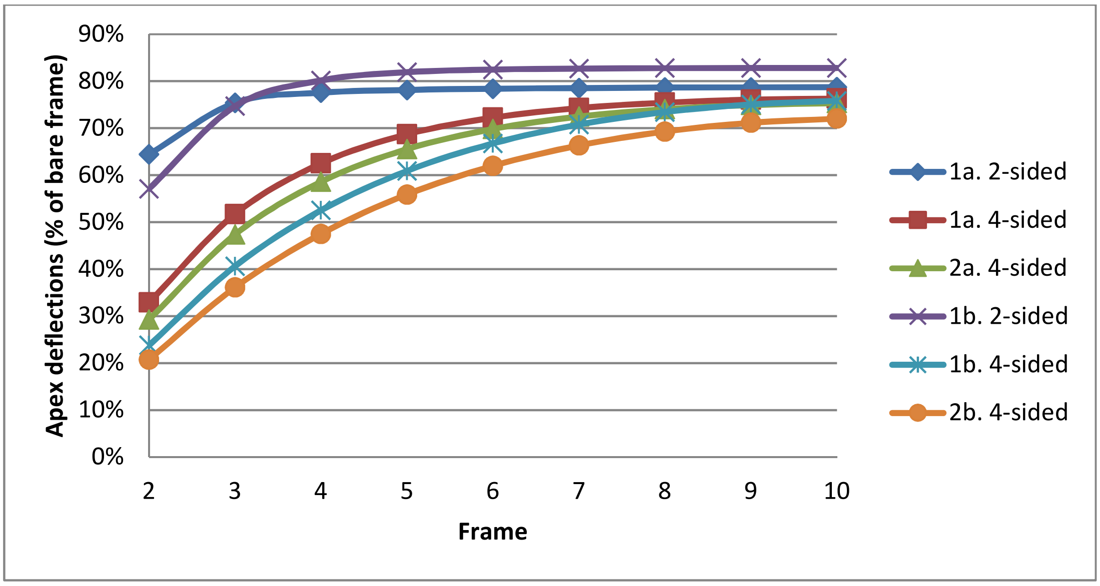


Figure 5.15 Apex deflections (Large building size, 2-bay)

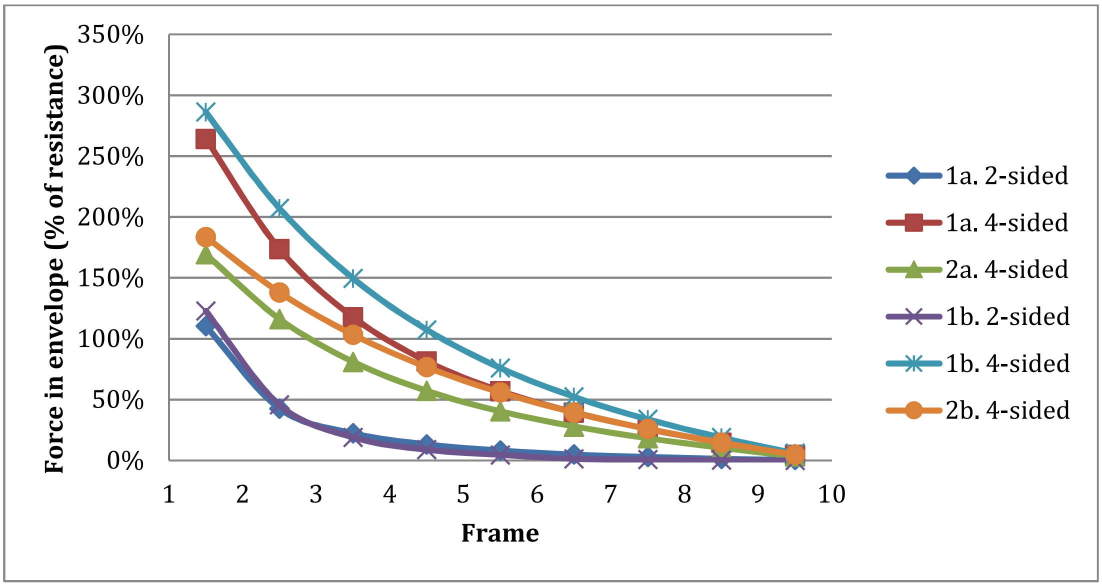


Figure 5.16 Force in the envelope – SLS (Large building size, 2-bay)

Note: End bay not normally critical due to presence of roof bracing.

The results indicate that the diaphragm resistance is exceeded for all the 4-sided schemes, even when the end bays are fully braced. This is regardless the roof pitch and span arrangements. Furthermore, the 2-sided arrangements show a considerable reduction of the deflections at eaves and limited at the apex. The 4-sided arrangements on the other hand, show a remarkable reduction of deflections at eaves and considerable at the apex. Finally, the range of deflection reduction was very similar across all the 4-sided arrangements regardless the roof pitch and spanning arrangement. The only exception was the 2-bay medium size building for which different effects of the various schemes are more obvious.

5.4.4 Scope for frame re-design and steelwork appraisal

Frame design with the aid of diaphragm action was performed in accordance with Section 7 in BS 5950-9:1994 which requires all frames in the building to be similar. Furthermore, Section 7 is only applicable for regular diaphragms and 1-bay rigid frames. The standard allows scope for re-analysing the frame to take account of the diaphragm action by using:

- Elastic analysis and accounting for the reduced forces, bending moments and deflections caused by the envelope restraining joint movements; for that case the reduction factors earlier calculated for the 1-bay cases and shown in Table 5.10 and Table 5.11 may be applied at each frame in the building to re-estimate the behaviour in sway and spread modes.
- 'Plastic' analysis accounting for the envelope reaching its plastic resistance and sustaining large plastic deformations at its design shear capacity. The frame can then be designed normally (either elastically or plastically). The effect of diaphragm action is to modify the loading on frames as shown in Figure 5.17 and Figure 5.18.

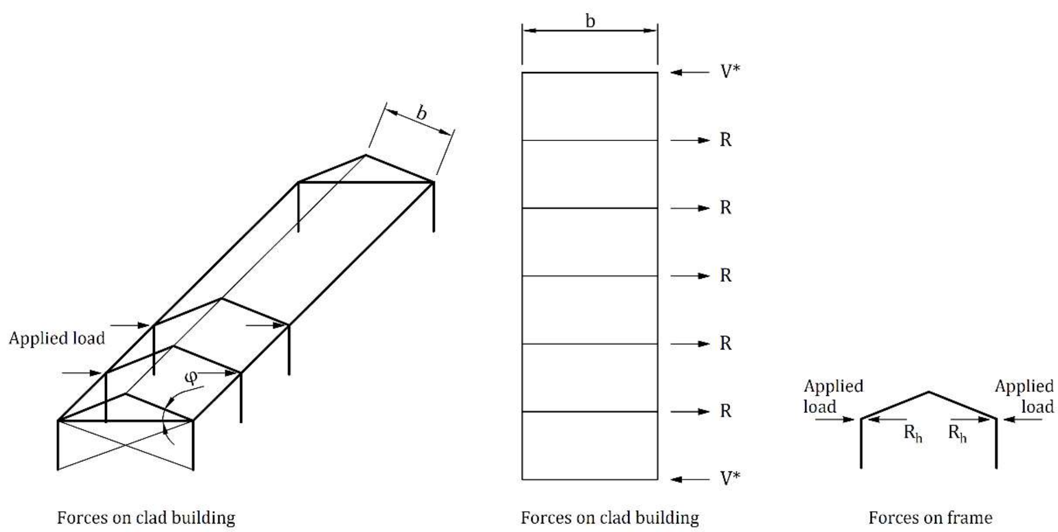


Figure 5.17 Plastic design of a clad pitched roof portal frame under side loads (adapted from BS 5950-9:1994)

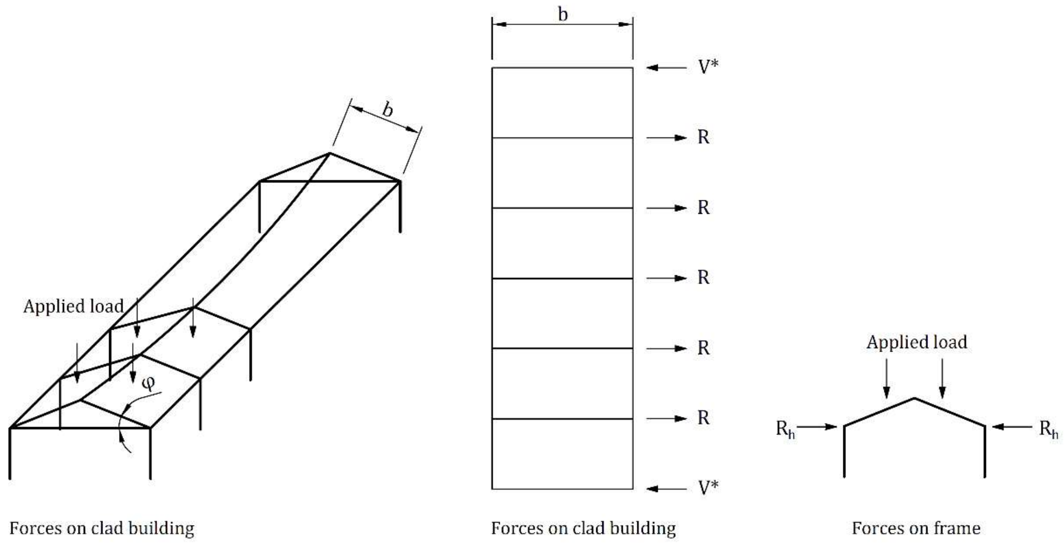


Figure 5.18 Plastic design of a clad pitched roof portal frame under vertical loads (adapted from BS 5950-9:1994)

The elastic analysis approach allows for modification of forces, bending moments and deflections. However, as a consequence of the frame similarity requirement, frame modification by the envelope's stiffness can only be allowed at the most onerous locations, i.e. for the intermediate frames. Any potential benefit arising for the other frames is then sacrificed to allow for similarity across the building. As it is easily noticed in Table 5.10 to Table 5.13, the load ratio for the intermediate frames is much higher compared to the ones for the penultimate frames (next to the gables), i.e. the effect of diaphragm action at those locations is much less.

Section 7 in BS 5950-9:1994 does only explicitly allow for frame re-design through the use of 'plastic' analysis. This approach is not identical to the conventional plastic analysis, since the formation of the first plastic hinge is not considered to occur in the frame rather in the envelope itself when it reaches its design shear resistance. In order for the plastic hinge to be sustained, the standard requires large shear deformations to be sustained, achieved through a ductile failure mode, such as seam fastener failure or panel / shear connector fastener failure, governing the panel's resistance. For the panel cases examined and shown in Table 5.3 and Table 5.4, the ductile mode of seam fastener failure was shown as dominant in all cases (except Scheme 1a for small building size where some additional end fasteners are required to avoid premature failure). At the ultimate load of the building, the collapse will then occur in all intermediate frames simultaneously. At this stage, the forces on each frame will be the same (see Figure 5.17 and Figure 5.18).

In order to permit frame variation across the building and optimal frame member sizing, full 3-dimensional frame analysis would need to be utilised according to Section 8 in BS 5950-9:1994. This approach would require finite element analysis and an iterative procedure to account for the modification of relative flexibilities and to allow calculation of the cladding's diaphragm action contribution to each frame which may then be dissimilar to the others. Tools for such type of analyses, however, require significant computational effort and resources which are not normally possessed by small and medium design consultancies. Since the present study is based on following the procedures normally met in practice, the simple approach in Section 7 in BS 5950-9:1994 was selected for the frame analysis.

Design of the frames was performed using the normal design process in Section 7 of BS 5950-9:1994 (referred to as 'plastic') to calculate the restraining forces due to the shear panels' resistance (as shown in Figure 5.17 and Figure 5.18) and then using BS EN 1993-1-1:2005 to elastically design the portal frames. This procedure would not require any special tools or software and would normally be followed in practice by a typical small or medium UK design consultancy. It was assumed that roof bracing would be present at the end panels as in normal practice, effectively reducing the length of the shear girder. The assessment was only performed for 1-bay frames, i.e. for the small and medium building sizes. BS 5950-9:1994 suggests that for 2-bay analysis 3-dimensional modelling is adopted. This would demand increased computational effort. As it will later be shown, the 1-bay analysis suggested very limited gains in terms of resizing structural members and reducing steelwork with the aid of diaphragm action. Consequently, it was decided that extending the analysis to 2-bay frames would not be necessary to draw further conclusions regarding advancing the opportunity, hence omitted.

A summary of the structural steelwork weights per unit floor area for the assessed structural schemes for the small and medium building sizes is shown in Figure 5.19 and Figure 5.20. The quantities include only the weight of the frames. This is in order to quantify the reduction in frame weight due to utilisation of the roof envelope's in-plane shear capability. Purlins, struts, roof and wall bracing weights are excluded from these charts. A breakdown of the portal frame steelwork weights for each scheme is shown in Table 5.14 and Table 5.15 together with a percentage reduction compared to the base case for each building size. A summary of the frames' design and the specified section sizes is given in Appendix D. The effect of varying building dimensions in the wind loading was taken into account when designing the frames and components.

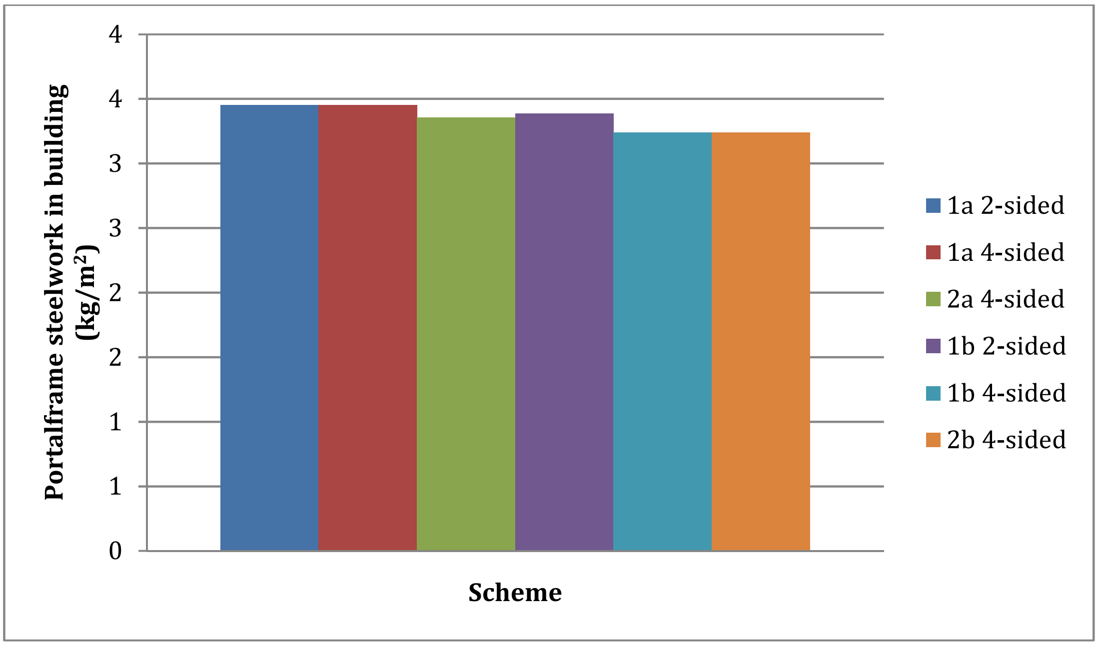


Figure 5.19 Summary of portal frame steelwork in 1-bay Small building

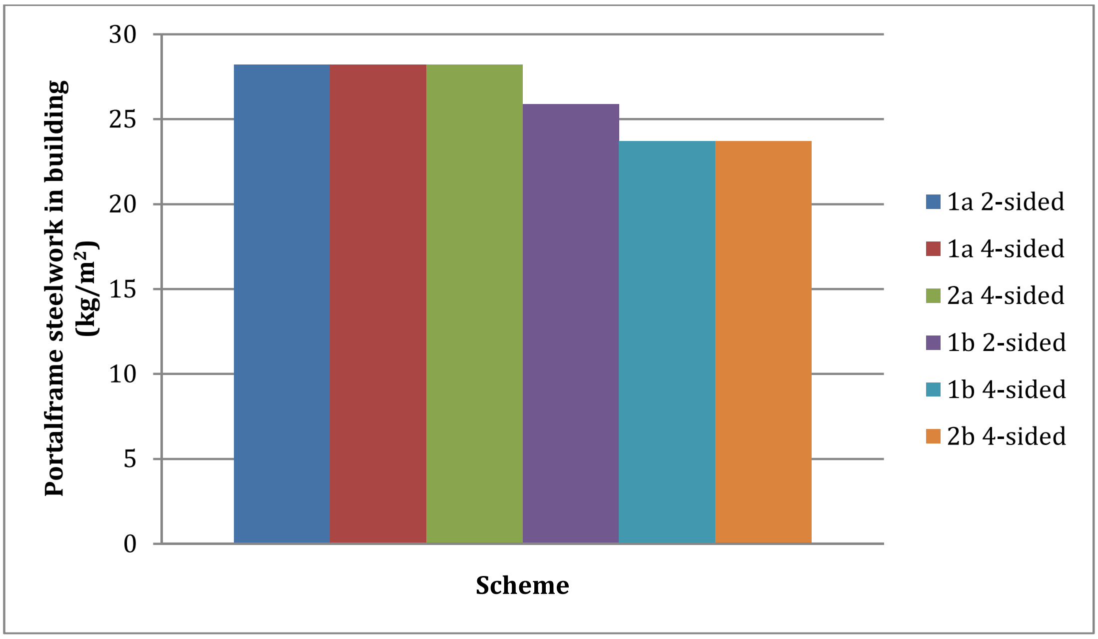


Figure 5.20 Summary of portal frame steelwork in 1-bay Medium building

Table 5.14 Breakdown of steelwork weight for portal frames with diaphragm action using the plastic resistance of the envelope (Small building, 1-bay)

Shear panel description	**Portal frame weight in building (tn)**	
	Normal roof pitch (6^{o}) (Scheme a)	High roof pitch (12^{o}) (Scheme b)
2-sided fastening, spanning between purlins (Scheme 1)	13.8 (base case)	13.5 (-2.2%)
4-sided fastening, spanning between purlins (Scheme 1)	13.8 (-)	12.9 (-6.5%)
4-sided fastening, spanning between rafters (Scheme 2)	13.4 (-2.9%)	12.9 (-6.5%)

Table 5.15 Breakdown of steelwork weight for portal frames with diaphragm action using the plastic resistance of the envelope (Medium building, 1-bay)

Shear panel description	**Portal frame weight in building (tn)**	
	Normal roof pitch (6^{o}) (Scheme a)	High roof pitch (12^{o}) (Scheme b)
2-sided fastening, spanning between purlins (Scheme 1)	112.8 (base case)	103.3 (-8.4%)
4-sided fastening, spanning between purlins (Scheme 1)	112.8 (-)	94.6 (-16.1%)
4-sided fastening, spanning between rafters (Scheme 2)	112.8 (-)	94.6 (-16.1%)

The results show that there is no real steelwork weight advantage as evidenced by Figure 5.19, Figure 5.20, Table 5.14 and Table 5.15. A small reduction in steelwork weight is achieved only for the high pitch roof scheme (b) (6.5% for the small and 16.1% for the medium size building). Furthermore, both 4-sided arrangements of schemes 1 and 2 have the same results. There is also a very small scope for steelwork reduction even for the as-built arrangement (2-sided, scheme1) for the high roof pitch.

In order to facilitate comparison with the long span opportunity, the total steelwork (i.e. frame, purlins, bracing systems, ties, struts) weight in the buildings for each diaphragm action option is shown in Table 5.16 and Table 5.17. The weights of the components other than the portal frames are identical to those shown in Table 4.6 and Table 4.7 for the relevant cases (1-bay portal frames with and without purlins at 6.67m frame spacing with 6^{o} roof pitch). For the high pitch roof the modifications to roof and wall bracing steelwork were calculated. Furthermore, the requirement for additional roof bracing to prevent premature failure of the envelope in shear as shown in Figure 5.7 and Figure 5.10 is noted where relevant in Table 5.16 and Table 5.17. With all components considered, it is still and even more evident that there is no meaningful steelwork weight advantage (9.6% for the small and 14.5% for the medium size building).

Table 5.16 Breakdown of total steelwork weight in building for portal frames with diaphragm action (Small building, 1-bay)

Frame type	Scheme	Shear panel arrangement	Spacing (m)	Steel total (tn)	Frame (tn)	Ties (tn)	Purlins (tn)	Roof bracing (tn)	Wall bracing (tn)	Struts* (tn)	Comparison to Base Case
Duo-pitch portal with purlins (1-bay)	1a	2-sided	6.67m	**17.8**	*13.8*	-	*2.1*	*0.9*	*0.4*	*0.5*	Base Case
		4-sided		**17.8**	*13.8*	-	*2.1*	*0.9*	*0.4*	*0.5*	-
	1b	2-sided		**18.1**	*13.5*	-	*2.1*	*1.1*	*0.5*	*0.5*	+1.7%
		4-sided		**17.1**	*12.9*	-	*2.1*	*1.1*	*0.5*	*0.5*	-3.9%
Duo-pitch portal without purlins (1-bay)	2a	4-sided		**16.3**	*13.4*	*1.0*	-	*0.9*	*0.4*	*0.5*	-8.4%
	2b	4-sided		**16.1**	*12.9*	*1.0*	-	*1.1*	*0.5*	*0.5*	-9.6%

**additional roof bracing at the penultimate bay would be required to avoid failure of the envelope*

Table 5.17 Breakdown of total steelwork weight in building for portal frames with diaphragm action (Medium building, 1-bay)

Frame type	Scheme	Shear panel arrangement	Spacing (m)	Steel total (tn)	Frame (tn)	Ties (tn)	Purlins (tn)	Roof bracing (tn)	Wall bracing (tn)	Struts* (tn)	Comparison to Base Case
Duo-pitch portal with purlins (1-bay)	1a	2-sided	6.67m	**126.6**	*112.8*	-	*7.3*	*2.2*	*1.1*	*3.2*	Base Case
		4-sided		**126.6**	*112.8*	-	*7.3*	*2.2**	*1.1*	*3.2*	-
	1b	2-sided		**118.3**	*103.3*	-	*7.3*	*2.4*	*1.5*	*3.2*	-6.6%
		4-sided		**109.6**	*94.6*	-	*7.3*	*2.4*	*1.5*	*3.2*	-13.4%
Duo-pitch portal without purlins (1-bay)	2a	4-sided		**125.2**	*112.8*	*5.9*	-	*2.2**	*1.1*	*3.2*	-1.1%
	2b	4-sided		**108.2**	*94.6*	*5.9*	-	*2.4**	*1.5*	*3.2*	-14.5%

**additional roof bracing at the penultimate bay would be required to avoid failure of the envelope*

5.5 Discussion

The results of the analysis showed the following:

- For frames with normal roof pitch, the governing failure mode was 'spread' under gravity loading. Since the beneficial effect of diaphragm action was only evident for the 'sway' mode, there was no or almost negligible scope for re-design of the frames. However, the diaphragm action resulted in a reduction in deflections, particularly those associated with the 'sway' mode.
- For frames with higher roof pitch, the governing failure mode was in a combination of 'spread' and 'sway'. Although diaphragm action had a greater contribution compared to the normal roof pitch, it was not enough to permit meaningful reduction in steelwork weight. The following frame weight savings were shown:
    - for the 'as-built' envelope arrangements (2-sided fastened spanning between purlins): 2.2% for the small and 8.4% for the medium building.
    - for the 'enhanced' envelope arrangements (4-sided fastened spanning between either purlins or rafters): 6.5% for the small and 16.1% for the medium building.

  When the total steelwork in the building is taken into account the maximum total savings reduce to 9.6% for the small and 14.5% for the medium-size building. The scope for reducing deflection was, however, significant.

- The 'as-built' envelope arrangement showed a remarkable reduction of the eaves deflections compared to the deflections calculated for a bare frame. The reduction of the apex deflection, however, was negligible, as the diaphragm action contribution is greater for the 'sway' mode of the frame rather the 'spread' mode for relatively low pitch roofs. Also, the developed forces within the roof envelope were found to be well below the in-plane shear resistance of the sandwich panel.
- The 'enhanced' envelope arrangement showed a remarkable reduction of deflections at both eaves and apex, as diaphragm action has an effect in both 'sway' and 'spread'. However, the shear forces exceeded the panel's in-plane shear resistance because the stiffer envelope attracted a higher force. Hence, additional roof bracing would be required to prevent premature failure of the

envelope under serviceability conditions. This is contrary to the aim of reducing the weight of the structure.

- Overall, it may be concluded that diaphragm action opportunities for single storey buildings with relatively low pitch roofs are limited to the reduction of deflections, while there is no real scope for meaningful reduction of structure weight. For frame cases where 'sway' loads and deflections govern the design, the scope for using diaphragm action to specify lighter member sizes may be more meaningful.

Furthermore, the following observations are highlighted as key for any future studies:

- Although buildings with higher pitch roofs are likely to demonstrate higher exploitation of diaphragm action, it is unlikely that roof pitches higher than 10^o are considered in the (UK) practice. This is because roof pitch increases are detrimental to operational energy use due to the increase of the building volume.
- There is a potential issue with the simple design approach in Section 7 in BS 5950-9:1994, which requires frame similarity across the buildings. Hence, frame modification is governed by the most onerous location, i.e. for the intermediate frame. If different frames were to specified to exploit diaphragm action further, the standard requires finite element analysis, a tool not normally possessed by small or medium UK consultancies. Furthermore, adopting dissimilar frames is largely avoided by steelwork contractors for repetition purposes.
- In addition, the simple approach in Section 7 in BS 5950-9:1994 applies only for 1-bay portal frames, whilst finite element analysis is required for multiple bays. As the 1-bay analysis demonstrated limited gains, it was not deemed necessary to extend the analysis to 2-bay to decide whether to advance the opportunity.
- The limited gains demonstrated in this study were for the assumed building geometries which are typical in the UK. Nevertheless, diaphragm action may be more significant for other geometries, such as buildings with smaller ratio of length / roof depth (similar to shear governing in shorter rather longer beams) or frames governed by 'sway' modes, e.g. with high ratio of frame height / width.
- There may be higher gains for plastically rather elastically designed portal frames, as the later use smaller section sizes. Consequently, such frames are more flexible and their flexibility relatively to the cladding is also lower. This

makes the diaphragm action contribution more significant. Also, due to the smaller section sizes, deflections may govern their design, hence their reduction with the aid of diaphragm action may lead to more meaningful steelwork savings.

- The study did not consider rooflight openings, typically at 12% of the roof area for energy conservation purposes. Openings in more than 3% of the roof area have very onerous effects in the diaphragm's strength and flexibility. However, as the study showed that there are no real benefits without openings, it was unnecessary to extend the analysis to include them. Any future considerations should include provisions for limiting such onerous effects of rooflights.
- The combined effects of in-plane shear and out-of-plane load on the metal sheets of sandwich panels should be considered as current literature is very sparse.
- Enhanced sandwich panel diaphragms were shown susceptible to premature failure as they attract more load due to their increased stiffness. Higher grade steels could be considered in the future, as they can increase the bearing resistance of the sheets, which governs the panel's in-plane shear resistance.

5.6 Decision-making on opportunity advancement

The study identified that for the typical building geometries specified in the present research, there is some small scope for structural material reduction with the aid of diaphragm action, quantified as up to 9.6% and 14.5% total steelwork reduction for the small and medium size buildings respectively. The most scope was found to be for high roof pitch portal frame buildings with the 4-sided fastened sandwich panels. For panels spanning directly between rafters the scope was somehow higher than those spanning between purlins. Regardless of the structure-envelope assembly scheme, these material reduction magnitudes are very limited when compared to the long span opportunity discussed in Chapter 4. Furthermore, the higher roof pitch is highly unlikely to be implemented in practice due to increase of the building volume and consequential onerous requirements for operational energy.

Furthermore, there is a series of technical issues which would require further examination if that opportunity was to advance. In specific:

- The 4-sided arrangements were susceptible to in-plane shear failure. This is because their increased stiffness led to attraction of higher percentage of the load distributed between the frame and the cladding. Hence, strength improvements for these arrangements would be required. Since resistance was mostly dominated by bearing failure of the steel sheets at the connections, the use of higher strength steel could increase the bearing strength of the fastening arrangements without modifying their stiffness.

- The study took no account of rooflights. That case would impose additional requirements to overcome reduction in strength and stiffness of the shear panels. Quantification of the opening effects on the diaphragm action response would be necessary, together with engineering provisions to limit any reduction in terms of strength and stiffness.
- The effects of combined in-plane shear and out-of-plane diaphragm action require further examination, particularly for the case of panels spanning between rafters. This is because for such arrangements panels are likely to be designed near their full capacity for out-of-plane loading.
- Code provisions may significantly limit the benefit shown by design, particularly due to the requirement for similarity of frames across the building. However, more sophisticated software and analytical methods such as 3-dimensional finite element analysis and iterative studies may allow for optimisation of frame specification and additional benefit in terms of reducing frame weight. Therefore, a more realistic analysis of diaphragm action effects with the aid of more sophisticated modelling methods would be required if the scope for frame optimisation was to be further examined.
- There is potentially more benefit for structures of different geometries (such as for small length/width and width/height ratios) in which shear girder effects, sway modes and deflection controls (i.e. areas in which diaphragm action is more significant) are more dominant in the design. Furthermore, diaphragm action may have higher benefit to plastically designed portal frames due to their higher flexibility (because of utilisng smaller member sizes) and susceptibility to defections, compared to those elastically designed. A more general parametric study would be required in order to determine the areas and range of meaningful diaphragm action applicability.

A summary of the scope for structural material reduction, the advantages and disadvantages and the further work required for each scheme is shown in Table 5.18. The table focuses on diaphragm action, while reference can also be made to the earlier assessment summary in Table 4.9 for the long span opportunity. The results and intermediate steps of the decision-making process are shown in Table 5.19. The decision whether to take each opportunity and scheme forward is made based on the decision-making process described in Chapter 3.

Overall, the study to evaluate the opportunities arising from diaphragm action showed that there is no significant scope for improvement in terms of structural efficiency of frames. This is based on the assumptions used in the study. There are compelling arguments that for the typical buildings examined in the present study the opportunity for material reduction is very limited, especially when compared to the long span opportunity discussed in Chapter 4, where the range of applications is wider and the scope for steelwork reduction significant with relatively small improvements to the current construction technology. Hence, it was decided that the diaphragm action opportunity was not taken forward for further research.

Table 5.18 Summary of steelwork reduction, advantages and disadvantages and further work required

Frame scheme	Roof pitch scheme	Envelope scheme	Steelwork reduction in building	Advantages	Disadvantages	Further work required / Addressing of technical barriers
1 Portal frame with purlins	a Normal 6°	2-sided	-	• Considerable reduction of deflections at eaves • Some reduction of deflections at apex	• No scope for steelwork reduction	• Assessment of rooflights implications on diaphragm behaviour • Engineering of rooflights provisions for strength and stiffness • Use of more sophisticated and elaborate methods to assess scope for larger buildings with multi-bay frames • Parametric studies to define range of buildings where scope of exploiting diaphragm action becomes significant • Improvements in sandwich panels and connections to achieve higher in-plane shear strength. Such as use of high strength steel sheets
		4-sided	-	• Significant reduction of deflections at eaves • Considerable reduction of deflections at apex	• No scope for steelwork reduction • Exceedance of envelope's in-plane shear strength	
	b High 12°	2-sided	Small: +1.7% Medium: -6.6%	• Considerable reduction of deflections at eaves • Some reduction of deflections at apex	• Very small scope for steelwork reduction • Significant increase of building volume and consequential operational energy requirements	
		4-sided	Small: -3.9% Medium: -13.4%	• Significant reduction of deflections at eaves • Considerable reduction of deflections at apex	• Exceedance of envelope's in-plane shear strength	
2 Portal frame without purlins	a Normal 6°	4-sided	Small: -9.6% Medium: -1.1%	• Significant reduction of deflections at eaves • Considerable reduction of deflections at apex	• Exceedance of envelope's in-plane shear strength • No additional benefit in terms of diaphragm action when compared to Scheme 1	• All the above plus: • Combined effects of in-plane shear and out-of-plane loading require further studying

Continues to next page

Frame scheme	Roof pitch scheme	Envelope scheme	Steelwork reduction in building	Advantages	Disadvantages	Further work required / Addressing of technical barriers
	b High 12^{o}	4-sided	Small: -8.4% Medium: -14.5%	• Significant reduction of deflections at eaves • Considerable reduction of deflections at apex	• Significant increase of building volume and consequential operational energy requirements • Exceedance of envelope's in-plane shear strength	

Note: Focus is purely on diaphragm action. Further reference for the assessment of the schemes can be made in Table 4.9

Table 5.19 Decision-making for diaphragm action opportunity and schemes

Frame scheme	Roof pitch	Envelope scheme	Scope for steelwork reduction in building	Building size applicability range	Risk of technical barriers not being addressed	Step change from current practice		Advantages overweighing disadvantages	Decision to advance opportunity
Portal frame with purlins (Scheme 1)	Normal 6^{o} (Scheme a)	2-sided	No.	Wide.	Low-Medium.	None		No.	No.
		4-sided	No.			Small			
	High 12^{o} (Scheme b)	2-sided	No.	Limited.		Medium			
		4-sided	Low.						
Portal frame without purlins (Scheme 2)	Normal 6^{o} (Scheme a)	4-sided	No.	Wide.	Medium.	Medium			
	High 12^{o} (Scheme b)	4-sided	Low.	Limited.		Large			

Chapter 6 Frameless buildings

The current chapter presents the studies undertaken to investigate the feasibility of engineering frameless construction with the aid of sandwich panels. It also presents the structural appraisal of the proposed schemes.

6.1 Structural frame schemes

Sandwich panels are mainly used as roof and wall coverings, together with applications for internal spacing arrangements such as ceiling and internal walls. The panels are mounted on the supporting structure (secondary or primary components). The load applied to the envelope (wind, snow, imposed, gravity) is then transferred to the supporting structure following the load path.

Frameless construction with the aid of sandwich panels may be feasible on the basis of the panels replacing the conventional structural members and being used as part of the load bearing frame without a primary substructure. This would necessitate:

- Wall components resisting the lateral out-of-plane forces and, additionally, the vertical forces (acting as columns) and the in-plane lateral forces (acting as wall diaphragms for strength and stiffness).
- Roof components resisting lateral out-of-plane forces and, additionally, the in-plane lateral forces (acting as roof diaphragms for strength and stiffness).

A schematic is shown in Figure 6.1.

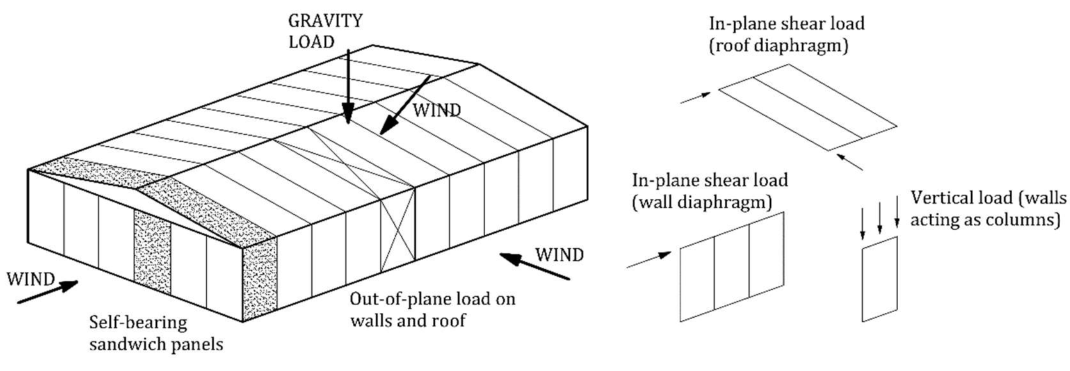


Figure 6.1 Frameless building schematic and actions on sandwich panel elements

Furthermore, the frame stability would rely on:

- The rotational capacity of the wall/roof junctions (and consequently the rotational capacity of the connections)
- The flexibility of the base supports

Finally, the presence of openings would lead to implications for the stability and resistance of the panel components and arrangements for out-of-plane and in-plane loading. At opening locations the panels may additionally be required to provide girder action.

Feasibility studies were undertaken to examine the resistance and stability of:

- Roof systems for out-of-plane and in-plane shear loading
- Wall systems for out-of-plane, in-plane vertical and shear loading

The aim of the investigations was to examine the suitability of sandwich panels for frameless applications and quantify any potential benefit. An assessment of the level of stress and resistance utilisation was, therefore, undertaken for a series of frameless building arrangements. Due to the feasibility nature of the study, the provisions for openings and the design of connections were appreciated but kept outside the scope of the investigation, so that the upper bound of the opportunity were examined.

6.2 Structural modelling and results

The current standard BS EN 14509:2013 for sandwich panels considers only applications that resist out-of-plane loading. The design of panels against in-plane shear and axial loading is not considered by the current design codes. In-plane resistance is covered by the recent European Recommendations by CIB-ECCS (2013).

The modelling procedure adopted for the present feasibility study was to follow the load path and assess each sandwich panel member or assembly in terms of stability and resistance. BS EN 14509:2013 was used to assess the panels in terms of out-of-plane loading as well as distribution of stresses within their cross-section. Guidance in the recent European Recommendations CIB-ECCS (2013) was used for the panel assessment against in-plane shear loading (for roof diaphragm's strength and wall diaphragm's stability and strength), together with the general modelling procedures in BS 5950-9:1994. Where required, a model deploying simple principles of structural mechanics was developed to assess the roof stability. Guidance by Käpplein and Missiek (2011g) and Käpplein and Ummenhofer (2010) were used for the assessment of the panels under in-plane axial load when acting as columns.

The details of the modelling procedure for each function are presented in the following sections together with the results in terms of resistance utilisation and stability.

6.2.1 Roof systems

For the purposes of the feasibility study, a typical roof sandwich panel system with modern specifications was assumed with the following geometry:

- 0.5mm external steel (S220) face with 32mm deep profiles and stiffened troughs
- 0.4mm internal (liner) lightly profiled steel (S220) face
- 135mm PIR core (ρ=38kg/m^3) to cover likely future thermal requirements for a notional building (U=0.15W/m^{2}K)

The full panel specification is given in Appendix A.

The system was then appraised for its ability to resist out-of-plane and in-plane loading and provide stability for various structural arrangements. These are explained and appraised below.

6.2.1.1 Out-of-plane loading

The size of the frameless buildings considered was limited by the spanning capability of the roof. This capability depends on the strength and stiffness of the panels along with the structural arrangements utilised. The following frameless roof arrangements were assessed:

1. Mono pitch roof (single-span panel)
2. Duo-pitch roof with pinned connections (two single-span panels)
3. Arched roof (curved single-span panel)

The following sections review each system in detail.

6.2.1.1.1 Mono-pitch roof system

When the panel is utilised as a mono-pitch roof system with pinned supports, its behaviour will be identical to a single-span one. A small roof slope (>3^o) is typically recommended by steel envelope manufacturers to avoid water penetration through the fasteners (TATA Steel, 2014). The implication of such a small roof slope to the structural behaviour of the panel in terms of the axial forces arising is almost negligible and is not normally taken into account even for conventional roof applications.

Modern roof sandwich panel systems such as the one described above and explained n more detailed in Appendix A may easily span up to 6.6m as a single-span arrangement and for the actions and load magnitudes utilised in this study (see Appendix B). Using the design methodology in BS EN 14509:2013 and the properties of the reference panel, the resistance utilisation for various spans is shown in Table 6.1. A full calculation example for the maximum allowable span is given in Appendix A.

6.2.1.1.2 Duo-pitch roof system

It may be assumed that, based on the spanning capability of single-span panel arrangements, pinned duo-pitch roofs with a span of up to 13.2m are within the limits of current technology. Longer spans would require sandwich panels with increased spanning capability.

As in the mono-pitch case, gravity loading on the roof gives rise to small axial force component. However, this is calculated to be negligible.

Table 6.1 shows the resistance utilisation ratios for various roof pitches when the axial load and second order effects are taken into account. It is easily noticed that the increase of utilisation due to the axial forces arising from the slope is almost negligible.

Table 6.1 Resistance utilisation for mono- and duo- pitch roof systems

Roof span (m)		**Resistance utilisation**
Mono-pitch	Duo-pitch	
5.0	10.0	50%
6.0	12.0	78%
6.6	13.2	100%

6.2.1.1.3 Arched roof

Curved sandwich panels provide another option for enhancing the spanning capability. This system is not available in the UK, but manufacturers of such assemblies exist in Europe (Berner, 2010b). The system requires a specialist production line, capable of rolling and forming the units in a curve, with a typical arch radius of 6-7m. Furthermore, special connection detailing would normally be required. Structurally, the system benefits from the arch action which facilitates the achievement of long spans, with the arch radius and height being the limiting parameters. Duo to the arching action, the bending moments are practically negligible and all loading is supported by axial forces developed in the plane of the panel.

A comparison between the resistance utilisation of a flat and a curved sandwich panel of 135mm core thickness is shown in Table 6.2, where a very low utilisation of the curved panels is evident. The stress distribution and resistance of the sandwich panels were modelled according to BS EN 14509:2013. The bending moments and axial loads in the truss system were calculated based on simple structural mechanics. It was assumed that under the axial forces developed in the arch, the stresses in the sandwich panel faces are distributed proportionally to their area. Also, column-buckling checks were performed according to the guidance of Käpplein and Ummenhofer (2010) for sandwich panels. The buckling length of the sandwich panel strut was estimated according to the guidance of King and Brown (2001) for curved steel beams.

Table 6.2 Resistance utilisation for flat and curved roof sandwich panels

Span (m)	**Radius (m)**	**Arch height (m)**	**Resistance utilisation**	
			Curved panel	Flat panel
6.0	7.0	0.68	7.3%	79%
8.0		1.26	7.8%	130%
10.0		2.10	8.4%	197%
12.0		3.39	9.1%	278%

It should be highlighted that despite the panels substituting the frame of the building, some steelwork has to be present at the panel junctions to create the conditions for adequate fastening and support. In particular, the steel plates used at the junction should provide adequate support width against core crushing and should possess adequate stiffness against fastener pull-out at wind uplift conditions. However, as shown in the design example in Appendix A, the performance of the reference panel in single-span conditions is dominated by deflections limit for downwards loading and by the resistance of the lower steel sheet for uplift. Hence, the support's steel gauge and width are not critical for the specified out-of-plane load conditions. The engineering of such detailing does not consist part of the present study as earlier explained.

6.2.1.2 Stability

The following roof system options may be used:

1. 3-pinned arch pitched roof (pinned at the apex and eaves), with the panels resisting both bending moments and axial loads
2. Flat roof acting as a simply supported beam
3. Curved roof acting as a simply supported beam
4. System with moment resisting connections at the eaves level, able to resist bending moments at the connections.

Based on the limitations of current technology, only Option 1 was considered in the current study. The use of Option 2 would severely limit the roof span, Option 3 is challenging in terms of production and Option 4 is not supported by current technology.

The stability of the 3-pinned arch pitched roof system relies on the stiffness of the supports, creating second order effects caused by the supports moving apart. The support stiffness may be provided by:

- The diaphragm action of envelope
- Installation of tie beams
- Column base fixity

A non-linear model was devised and a second order analysis was performed to assess the stability of a 3-pinned arch for the three aforementioned support options. Table 6.3 shows the results of the analysis in terms of required roof pitch so that global stability is provided.

Table 6.3 Required roof pitch for global roof stability for various support options

Panel length	**Support stiffness**	**Pitch required**
6.25m	Diaphragm action (2-sided fastened)	7° (ULS) 9° (SLS)
	Diaphragm action (4-sided fastened)	3° (ULS) 3° (SLS)
	Tie (SHS 25x25x2)	5° (ULS) 6° (SLS)
	Column base fixity	28° (ULS) 52° (SLS)

The results show that the diaphragm action and tie beam options are able to provide adequate support stiffness for the stability of 3-pinned arch roof. The option of accounting for the limited fixity of column would require unacceptably high roof geometries. Both diaphragm and tie-beam systems would require the engineering of suitable connection details. This requires further work outside the scope of the present study.

6.2.2 Wall systems

Sandwich panel wall systems for frameless construction would be required to resist not only the conventional out-of-plane wind loads but also the vertical loads transferred from the roof (acting as load-bearing walls) and in-plane shear loads (acting as wall diaphragms). A structural appraisal was, therefore, undertaken to assess the capability of the panels against the aforementioned load cases.

For the purposes of the feasibility study two wall sandwich panel system with modern specifications were assumed with the following geometries:

Wall panel type 1 – Lightly profiled panel with PIR core:

- 0.7mm external micro-ribbed steel (S220) face
- 0.4mm internal (liner) lightly profiled steel (S220) face
- 120mm PIR core (ρ=38kg/m^3) to cover future thermal requirements (U=0.17W/m^{2}K)

Wall panel type 2 – Flat panel with Mineral Wool core:

- 0.6mm external flat steel (S220) face
- 0.45mm internal (liner) flat steel (S220) face
- 200mm Mineral Wool lamella core (ρ=120kg/m^3) to cover future thermal requirements (U=0.20W/m^{2}K)

The full panel specifications are given in Appendix A.

6.2.2.1 Out-of-plane loading

Modern sandwich panels systems are capable of resisting high wind load magnitudes at relatively long spans (typically up to 8m). For small frameless buildings, a height of 3m-4m may be assumed to be representative and may be easily accommodated by modern sandwich panel assemblies. Table 6.4 shows the utilisation ratios for the two panel types and for 3m and 4m wall heights, where it is evident that these ratios are very low. The resistance of the panels was calculated according to BS EN 14509:2013.

Table 6.4 Resistance utilisation of sandwich panel systems under out-of-plane loading

Wall panel type	Wall height	Resistance utilisation	
		(positive)	(negative)
1	3m	17.8%	24.7%
	4m	29.9%	43.9%
2	3m	29.9%	32.5%
	4m	39.9%	43.3%

Assuming 3 fasteners per panel end and 100mm support width

6.2.2.2 Vertical loading (panels acting as columns)

Stresses are developed due to the vertical axial load and second order effects arising from:

- imperfections
- out-of-plane deflections (from wind and temperature effects)
- eccentricities at the end supports.

The panels would also be susceptible to creep due to permanent dead load apart from their self-weight. This aspect, however, was kept outside the scope of the present study.

A calculation was undertaken to assess the capability of the two sandwich panel wall systems to act as columns in terms of stability and resistance. To this end, the bending moments developed due to the axial load and the second order effects were converted with the use of an amplification factor similar to column-buckling into axial forces acting in the plane of the faces. The stresses developed from out-of-plane loading were also taken into account. This method has previously been adopted by Berner (2010a) and Käpplein and Ummenhofer (2010). The stress distribution and resistance of the panels was modelled according to BS EN 14509:2013. The initial imperfection was taken as height/500 according to BS EN 1993-1-1.

Furthermore, the resistance of the panel's faces against crippling (local stability failure of the faces), due to the load introduction to either one or both panel faces, was assessed based on Käpplein and Ummenhofer (2010).

The results are summarised in Table 6.5, showing the utilisation of the panel's resistance for each case.

Table 6.5 Resistance utilisation of sandwich panel systems acting as load-bearing walls (stability and resistance)

Wall panel type	Wall height	Roof span	Resistance utilisation		
			Axial	Crippling – Load through 1-face	Crippling – Load through 2-faces
1	3m	12.5m	17.6%	59.9%	5.8%
	4m		34.5%		
2	3m		18.9%	63.7%	11.1%
	4m		37.2%		

The results showed that a very low utilisation of the panel's resistance (lower than 38%) may be achieved for panels acting as load-bearing walls and resisting combined axial and out-of-plane loading. Furthermore, low utilisation (lower than 6%) against crippling of the faces was evident when the load was introduced at both panel faces. On the other hand, when the load is introduced through only one of the faces, a much higher utilisation ratio (up to 64%) was shown. Both sandwich panel options were fairly similar in terms of performance.

6.2.2.3 In-plane shear loading (panels acting as diaphragms)

Sandwich panel walls would be required to provide adequate lateral stability and resistance through diaphragm action. The use of goal-post style vertical and horizontal beams to connect panels at top, bottom and edges would also be required. Furthermore, seam fastening would ideally be needed in order to strengthen the diaphragm assembly. The use of intermediate vertical posts may provide further stiffness to the diaphragm.

An analysis was undertaken based on the principles of BS 5950-9:1994 to investigate the number of diaphragms and corresponding stiffening posts required across each building side for two different building sizes. Seam fastening was assumed at the longitudinal edges of the diaphragms. The results are summarised in Table 6.6.

Table 6.6 Appraisal of sandwich panel systems acting as diaphragm walls

Roof span / Building width	Building Length	Panel spanning arrangement	No. intermediate posts at	
			Gable walls	Longitudinal walls
12.5m	20.0m	Vertical	2	1

The results showed that sandwich panel wall diaphragms may provide adequate strength and stiffness for frameless buildings with the addition of only a few stiffening posts, particularly at the gable walls.

6.3 Discussion

The results of the analysis indicate the following:

- The size of frameless buildings is limited by the spanning capability of the roof. Modern roof sandwich panels may span up to 6.6m, while engineering of new systems would be required for longer spans. Consequently, mono-pitch roof systems up to 6.6m and duo-pitch roof systems up to 13.2m can be achieved as upper limits of the current sandwich panels spanning capability.
- Mono-pitch and duo-pitch roofs with sandwich panels acting as primary structure would require them to resist not only out-of-plane loads, but also axial in-plane loads occurring due to the roof pitch. The analysis showed that such axial loads have a very small effect and increase the utilisation of the panels only little.
- Arched panels may easily reach spans of 13m for a reasonable curve radius of 7m, exhibiting very low resistance utilisation ratios due to arched action and the consequential zero bending moments. The curve radius would also result in increased arch heights (3.4m for 12m span), leading to a greater building volume which is not necessarily desired and would demand higher operational energy requirements. Finally, engineering of arched panels would pose very challenging production issues and significant alterations and additions to the current continuous flat panel production lines. It is unlikely that sandwich panel manufacturers would invest on such technology unless a very wide range of applications was evident, which is not the case.
- For 3-pinned arch roof systems, the stability would rely on the stiffness of the supports. Increased support flexibility would lead them to spread apart under the load application, and give rise to second order effects and eventually collapse. The supports should, therefore, be adequately restrained against second order effects. This may be achieved with the aid of the diaphragm action of the roof assemblies themselves or by introducing ties struts. Exploiting the potential fixity of sandwich panel walls was found to require an unreasonable roof pitch for stability (52^{o}). For a 2-sided fastened roof diaphragm, a reasonable roof-pitch of 9^{o} was found to guarantee stability, while for a 4-sided fastened diaphragm a roof pitch as low as 3^{o} would be adequate. Introduction of tie-struts would also lead to a desirable roof-pitch of 6^{o}. The use of diaphragm action for roof stability is an approach very similar to the folded plate roof.

- For wall panels acting as load-bearing walls in a very small duo-pitch industrial building of 12m/8m width and 3m/4m height, very low utilisation ratios were exhibited (less than 40%) under in-plane axial and out-of-plane loads combined and associated second order effects. The results indicate that sandwich panels could be excellent means of resisting vertical loading. Careful consideration should also be given to the load application areas. Introducing the load through both faces of the panels is recommended as this leads to very small utilisation ratios (less than 10%) against crippling of the faces. Introducing the load only through one face shows high utilisation ratio (up to 71%) and should, therefore, be avoided.
- For the walls to adequately act as diaphragms against in-plane shear loading, additional stiffening with steel posts would be required. This is particularly required at the gable walls where high wind loads transferred from the longitudinal surface of the building need to be resisted.
- The study did not take into account effects of openings (such as for rooflights and windows) which are anticipated to have a considerable impact on the stiffness and strength of the diaphragms, as well as the resistance against out-of-plane loads.
- Engineering of connection details was outside the scope of the study.
- Combined stresses arising from in-plane and out-of-plane loads were not examined for the roofs due to lack of robust quantification methods. It is likely that roof systems would operate close to their maximum spans, hence capacities, and impacts of additional in-plane stresses need to be quantified.

6.4 Decision-making on opportunity advancement

A summary of the scope for structural material reduction, the advantages and disadvantages and the further work required for frameless buildings is shown in Table 6.7. The results and intermediate steps of the decision-making process are shown in Table 6.8. The decision whether to take each opportunity and scheme forward is made based on the decision-making process described in Chapter 3.

Overall, the study to evaluate the opportunities arising for frameless buildings by substituting the primary structure with sandwich panels showed that there is significant scope for steelwork elimination; however, the size of frameless buildings would be highly limited by the spanning capability of the sandwich panels. Within the limits of the current sandwich panel technology, frameless design could be applicable for very small buildings with roof spans up to 13.2m.

A number of technical issues would require further examination if the opportunity for frameless construction were to be advanced, specifically:

- Connection details need to be engineered, taking into account load introduction effects (such as crippling), connection flexibilities and rigidities and thermal bridging requirements.
- Provisions for openings (such as for rooflights and windows) need to be engineered, considering strength and stiffness aspects. Openings would have a negative effect on stiffness and strength of both wall and roof diaphragms, hence an assessment of their implications on the global and local stability of the panels and frameless building assemblies is required.
- The behaviour of sandwich panel wall diaphragms requires more experimental investigation. While theoretical solutions exist, these are not fully validated by testing up to now.
- The combined effects of in-plane shear and out-of-plane load for roofs have not been examined. While for conventional metal cladding a mere calculation of combined stresses in the sheets is adequate to allow appraisal of the effects, for sandwich panels a more elaborate analysis and testing is required. It is very likely that roof sandwich panels will operate near their maximum capacity for out-of-plane loading, leaving little scope to resist in-plane stresses.
- A more elaborate model (such as finite element analysis) of the whole structure and taking into account connection rigidities would be beneficial in terms of quantifying load distribution adequately.

The implementation of the frameless buildings opportunity would require a significant amount of further research and would be a big step changes from the current practice while its applicability would be very limited. The opportunity is also much narrowed compared to the long span opportunity discussed in Chapter 4, where the range of applications is wider and the scope for steelwork reduction is also significant with relatively small improvements to the current construction technology. Hence, it was decided that the frameless opportunity was not taken forward for further research.

Table 6.7 Summary of steelwork reduction, advantages and disadvantages and further work required

Frame scheme	Roof scheme	Steelwork reduction	Advantages	Disadvantages	Further work required / Addressing of technical barriers
Frameless buildings	Pitched roof (pinned ends)	Full frame elimination. Some steelwork only for connections and goal posts.	• Feasible construction. • Diaphragm action adequate to provide roof stability. • Sandwich panel columns are adequate to resist axial loads with very low utilisation ratios. • Excellent crippling resistance for load introduction through both faces. • Wall diaphragms adequate to resist in-plane shear loads.	• Building size is limited by spanning capability of the roof. • Wall diaphragms would require stiffening with intermediate posts, particularly at gable walls. • Effects of openings, such as for rooflights and windows, would be significant in terms of reducing the shear strength and stiffness of the diaphragms, as well as the out-of-plane load resistance of the panels.	• Engineering of connections, accounting for load introduction, connection flexibilities and thermal bridging. • Engineering of opening provisions. • Assessment of openings implications in sandwich panel behaviour for in-plane and out-of-plane loads. • Further structural testing required particularly for use of panels as wall diaphragms. • Assessment of combined stresses from out-of-plane and in-plane loading should be undertaken, comprising more elaborate analysis and structural testing. • Use of more elaborate modelling methods such as finite element analysis for the whole structure to take into account connection flexibilities and assess global stability.
	Arched roof	Full frame elimination. Some steelwork only for connections and goal posts.	All the above plus: • Long roof spans can be achieved with a reasonable curve radius of 7m.	All the above plus: • Challenging production issues • Increased arched height required, leading to significant additional building volume.	All the above plus: • Development of arched sandwich panels production line.

Table 6.8 Decision-making for frameless buildings opportunity and schemes

Frame scheme	Roof system	Scope for steelwork reduction in building	Building size applicability range	Risk of technical barriers not being addressed	Step change from current practice	Advantages overweighing disadvantages	Decision to advance opportunity
Frameless buildings	Pitched roof (pinned ends)	High.	Limited.	Medium.	Large.	No.	No.
	Arched roof	High.	Limited.	High.	Large.	No.	No.

Chapter 7 Optimised long span sandwich panels

The feasibility studies presented in Chapter 4, Chapter 5 and Chapter 6 concluded that the greatest opportunity for frame material reduction is for buildings with long-span roof systems, particularly trussed-roof frames with northlights. As discussed in Chapter 4, these were found to provide the greatest potential for steelwork reduction for an optimum frame spacing of 6.67m for small buildings and 8.00m for medium and large buildings. Currently available roof sandwich panels can already span 6.67m for the load magnitudes used in this study. The focus of the investigations presented in the current chapter is to specify the long-span panel forms to achieve the 8.0m span required for the medium and large buildings, while keeping embodied carbon emissions to a minimum. The current chapter presents the analytical and experimental studies undertaken to design an embodied carbon-optimal sandwich panel to achieve the required span distance and also to quantify its associated carbon emissions.

7.1 Aim and methodological approach

7.1.1 Aim

Sandwich panels comprise two materials with very dissimilar properties. The geometrical and material properties for each material and layer influence the behaviour of the whole panel. Consequently, the desired performance can be achieved by a large number of combinations, which all lead to different embodied carbon emissions. For the spans which were defined as optimum for the frame spacing in Chapter 4, an optimisation exercise was set to define sandwich panel specifications which would allow the applied loads to be resisted while yielding minimum embodied carbon emissions for the panel. The simultaneous increase in strength and reduction in embodied carbon is a problem with two competing objectives. The results can be used to theoretically demonstrate the feasibility of the desired structural performance and to determine the associated embodied carbon. These will later be used in Chapter 8 for the holistic building review.

The problem focused on fully profiled sandwich panels with steel faces and PIR cores, which represents the current practice for roof elements. In theory, different panel forms could also be considered. However, the focus of the study was on defining solutions with small step changes from existing technology. These would also be more likely to be implemented in practice.

7.1.2 Methodological approach

The adopted methodological approach was based on the concept of the methodology developed previously by Kurpiela (2013). This was to determine specifications of lightly profiled wall sandwich panels with steel faces and PUR cores for increased spanning capabilities and reduced costs. Kurpiela (2013) investigated the critical PUR core parameters (core density, core depth and homogeneity, temperature) which influence the mechanical properties of the core and the resistance of the sheets to suggest improvements in theory and develop mathematical models. These were then used to carry out a Pareto-optimisation exercise and define optimal solutions in terms of span and costs.

In the present research, improvements in sandwich panels are pursued to achieve long span performance. Section 7.2 discusses how the performance of the sandwich panel relies on the geometrical and mechanical properties of the steel sheets and the core. Should improvements in the long span performance of sandwich panels occur, these may be achieved by adjusting the geometrical and mechanical properties of the steel and core layers. Hence, an initial set of studies was undertaken to investigate the reliability and influence of the PIR cores and sheet sheets in the performance of the panel together with an assessment of the reliability and conservatism of current theoretical methods. The results of these investigations were then included in the optimisation model to determine the embodied carbon-optimal sandwich panel forms which address the span and load requirements identified in this study.

An analytical and experimental investigation was initially undertaken to examine:

- The influence of the PIR density on the mechanical properties of the PIR-core and identification of Poisson ratio for PIR.
- The influence of the steel sheets' mechanical and geometrical properties on the compressive resistance of the steel sheets and the bending capacity of the panel.

The test results and observations were then used to:

- Establish mathematical relationships for the calculation of mechanical properties of the core with varying values of density.

- Assess the conservatism of the currently available design guidance with regards to quantifying the design resistance of the sheets and select the methods with the highest reliability.
- Assess the impact of the material properties reliability on the design performance and suggest areas for future improvements.

A Pareto-optimisation process was then applied to theoretically define embodied-carbon optimal panel specifications for the defined applied loads. Design and manufacturing constraints were taken into account. The structural analysis of the panels was based on established design methods in BS EN 14509:2013, the analytical guidance selected to quantify the compressive resistance of the sheets and the developed mathematical relationships for the mechanical properties of the core. The selected methodology for the embodied carbon quantification was according to the University of Bath's Inventory of Carbon and Energy (Hammond and Jones, 2008).

The investigations are presented in the following sections within the current chapter.

Section 7.2 presents a review of the principles of structural behavior and the resistance of fully profiled sandwich panels. These are used in the analytical and experimental studies discussed within the current chapter.

Section 7.3 presents the investigations into the mechanical properties of the PIR core with regards to the influence of the PIR density and establishes mathematical relationships.

Section 7.4 presents the investigations into the influence of the steel sheets' mechanical and geometrical properties on their achieved compressive resistance and the reliability of the design guidance.

Section 7.5 presents an assessment of the impact of the material properties reliability on the design performance of long span sandwich panels.

Section 7.6 presents the development of the optimisation model and the results of the analysis and discusses the influence of various panel specifications on the load-carrying capacity and embodied-carbon emissions of the sandwich panels. The embodied-carbon optimal solution to be used in the later studies is also presented.

7.2 Principles of structural behaviour

7.2.1 Principles

The structural performance of sandwich panels relies on composite action between the core and the metal sheets. The external load applied to a sandwich panel causes the development of internal stresses which are assumed to be divided into two load-carrying systems, namely the 'flange' and the 'sandwich' part (Stamm and Witte, 1976). The 'flange' part refers to the bending resistance and stiffness of the steel sheets about their neutral axis, while the 'sandwich' part refers to the bending action about the neutral axis of the sandwich panel. The 'flange' contribution depends only on the geometry and material properties of the two faces, while the 'sandwich' contribution also depends on the depth and properties of the core. The 'sandwich' part distributes the applied load as axial forces in the steel faces while it simultaneously resists the developed shear force. The separation of the load-carrying systems and the distribution of stresses are illustrated in Figure 7.1.

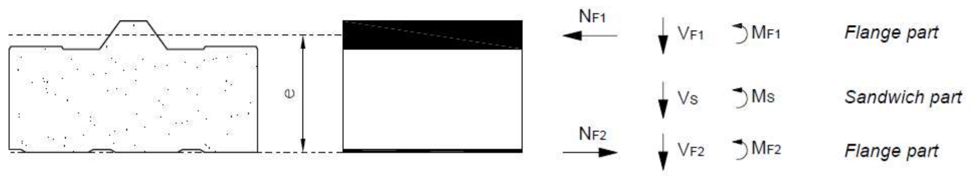


Figure 7.1 Separation into 'sandwich' and 'flange' part (Adapted from EN 14509:2013)

The mathematical basis of the analysis of profiled sandwich panels have been developed by Stamm and Witte (1976) on the basis of Plantema (1966) and Allen (1969) and expanded by Davies (1986, 1987) and Berner (1998). The most comprehensive design manual to date is available by Davies *et. al.* (2001).

BS EN 14509:2013 is the British Standard for the design, testing and specification of sandwich panels and is largely based on Davies *et. al.* (2001) and early guidance in the 'European Recommendations for Sandwich Panels' (CIB, 2000). The standard is not exhaustive and reference to the manual by Davies *et. al.* (2001) is made for complimentary guidance. The provisions of BS EN 14503:2013 are used for the specification of sandwich panels within the current exercise together with additional design guidance where required.

7.2.2 Failure modes

For single-span sandwich panels in flexure, the following failure modes are possible:

- Flexural wrinkling and / or local buckling or debonding of the sheet in compression
- Yielding of the sheet in tension
- Shear failure of the core
- Compression failure of the core at the supports.

Other modes such as shear failure of the sheet's webs are not common. For panels with sprayed PIR foams, debonding is not a common type of failure either.

BS EN 14509:2013 provided design guidance for the calculation of stresses and resistances for the various failure modes. Full expressions of the stresses and resistances are given in the standard. A statistical process is also required in order to address deviation of the values derived by testing. In the current chapter, the following notation is used for strength and stress in sheets and cores:

- Stress in outer sheet: σ_{F1}
- Stress in inner (liner) sheet: σ_{F2}
- Shear stress in core: τ_C
- Compression stress in core at support: σ_{Cc}
- Deflection at mid-span: w
- Strength of sheet in tension: f_y
- Strength of sheet in compression: σ_w
- Strength of core in shear: f_{Cv}
- Strength of core in compression: f_{Cv}

For sheets in compression, the following failure modes are possible, depending on the sheet type (Pokharel and Mahendran, 2004, 2005, Davies *et. al.*, 2001):

- For flat sheets: flexural wrinkling
- For fully profiled sheets: local buckling
- For lightly profiled sheets: local buckling followed by flexural wrinkling (mixed mode)

Flexural wrinkling is a form of instability observed in flat sheets where the large sheet (plate) widths are unstiffened but stabilised by the core. Fully profiled sheets have short plate widths which are well-stiffened by the full profiles and stabilised by the core, exhibiting significant reserve of strength and failure in local buckling in a ductile mode. Lightly profiled sheets also have short plate widths; however the short ribs have less stiffening effect compared to the full profiles. Lightly profiled sheets exhibit initial failure as local buckling and once the first buckles are formed flexural wrinkling follows as instability failure.

7.2.3 Influence of sheet and core properties on sandwich panel performance

The sandwich panel assemblies intended for long span applications would be required to act primarily in bending. Their resistance and stiffness would depend on both the steel sheet and PIR insulation core materials as well as on their interdependence. In particular:

- The mechanical properties of the core influence the bending and shear stiffness of the panel, the shear and compressive resistance of the panel near the supports and the resistance of the steel sheets against compressive axial load through the core's stabilising function.
- The depth of the core influences the panel stiffness, the bending and shear resistance through the internal force distribution.
- The geometry and steel strength of the sheets influence the stiffness of the panel, as well as the resistance of the steel sheets in tensile and compressive axial load and the bending resistance overall.
- The bond between the skin and the core is crucial for the shear transfer when the panel is in bending.

In order to calculate the resistance of a sandwich panel with steel sheets in bending, the following mechanical properties are required to be determined:

- G_C: core shear modulus
- f_{Cv}: core shear strength
- f_{Cc}: core compression strength
- f_y: steel yield strength
- σ_w: compressive strength of the steel faces, depending on the following mechanical properties:
    - G_C, f_y
    - E_{Cc}: core compression modulus
    - E_{Ct}: core tension modulus

BS EN 14509:2013 requires that the sandwich panel resistance is derived by testing. Large scale bending tests are prescribed in BS EN 14509:2013 for the calculation of the bending moment capacity of panels and the resistance of the metal sheets in compression. A theoretical model is made available for the calculation of the compressive strength of the sheets. However the model is meaningful only for flat sheets, while provides extremely conservative results for the compressive resistance of fully and lightly profiled sheets and, therefore, should be avoided for those calculations.

For lightly profiled sheets in compression, previous research and design guidance has been developed. For the needs of the current exercise, the design guidance by Pokharel and Mahendran (2004), CIB (2000) and Davies *et. al.* (2001) is used to compare with test data. For fully profiled sheets the design guidance by Pokharel and Mahendran (2005) is used. More details are provided in 7.4.

7.3 Investigation of PIR core mechanical properties

As discussed in Section 7.2, the core influences the global structural response of the panel in terms of shear resistance and stiffness, as well as the compressive resistance of the sheets through its stabilising function. Hence, the scope for improvement of the sandwich panel performance depends on improving the reliability and the mechanical properties of the core. The mechanical properties of PIR cores are reliant on the following factors (Kurpiela, 2013, Kurpiela and Lange, 2013, Davies *et. al.*, 2001):

- Foam cell structure
- Foam homogeneity (Kurpiela, 2013, Hassinen and Misiek, 2012)
- Temperature
- Foam density

High temperatures reduce the tension modulus of the core and consequently may deteriorate the compressive resistance of the sheets. BS EN 14509:2013 requires that the tensions modulus in high temperatures is derived by testing and a formula for the reduction of the compressive strength of the sheets is provided. The cell structure influences the core's mechanical properties, which is evident for different materials (PIR, PUR, EPS etc.) and also the properties in different direction (orthotropic, isotropic or anisotropic). The homogeneity of the core influences how the mechanical properties are distributed across the core specimen and consequently influence the resistance of the sheets and the global behavior of the panel. The foam density influences the mechanical properties of the core. Typically, the denser the PIR foam the higher the moduli and strength of various mechanical properties. The foam density is also an important parameter for the embodied-carbon optimisation exercise in Section 7.6, since the denser the foam the higher the embodied carbon.

The influence of the cell structure, the homogeneity of the core and temperature effects were outside the scope of this research exercise. The investigation of the foam density on the mechanical properties is investigated in the current section.

The influence of the core properties on the compressive resistance of the sheets was examined separately and this is presented in Section 7.4.3.

7.3.1 Influence of PIR core density on its mechanical properties

The core density has a direct effect on its mechanical properties and consequently on the structural behaviour of the sandwich panel. An investigation was carried out to develop mathematical relationships between density and mechanical properties of typical PIR cores used for sandwich panels. This would later allow relating sets of mechanical properties to a single parameter, being the core density, to be used for design. This would significantly reduce the size of the optimisation problem. This methodology was previously adopted by Kurpiela (2013) and Kurpiela and Lange (2013).

The influence of the core's PIR density on its mechanical properties was investigated based on test data provided directly by manufacturers. These were derived according to the testing procedures in BS EN 14509:2013 as part of the routine quality control tests at the production line. The data comprised variations against the following parameters:

- Core density
- Insulation depths

The data set comprised test results for the following mechanical properties: E_{Cc}, E_{Ct}, G_C, f_{Cc}, f_{Ct}, f_{Cv}.

The range of density values was 35kg/m^3 to 42kg/m^3.

The range of insulation depths of the specimens was 70mm to 120mm. The variation between the mechanical properties across the three insulation depths was found to be small. Moreover, the influence of the insulation depth on the property values could not be conclusive. Hence, it was decided not to differentiate further between insulation depths.

A statistical analysis was applied to the measured data to derive mathematical relationships between mechanical properties and the PIR foam density. These were developed based on a first order trendline for the data set. This approach was previously used by Kurpiela (2013). Very few values were found to be lower than the 5% fractile value. These were eliminated from the data set, as the specimens would normally be rejected in practice by the manufacturer as part of their quality assurance process.

An indicative illustration of the measured data plots and the linear trendline to established mathematical relationships is shown in Figure 7.2. The full measured data are shown in Figure E.1 and Figure E.2 in Appendix E.

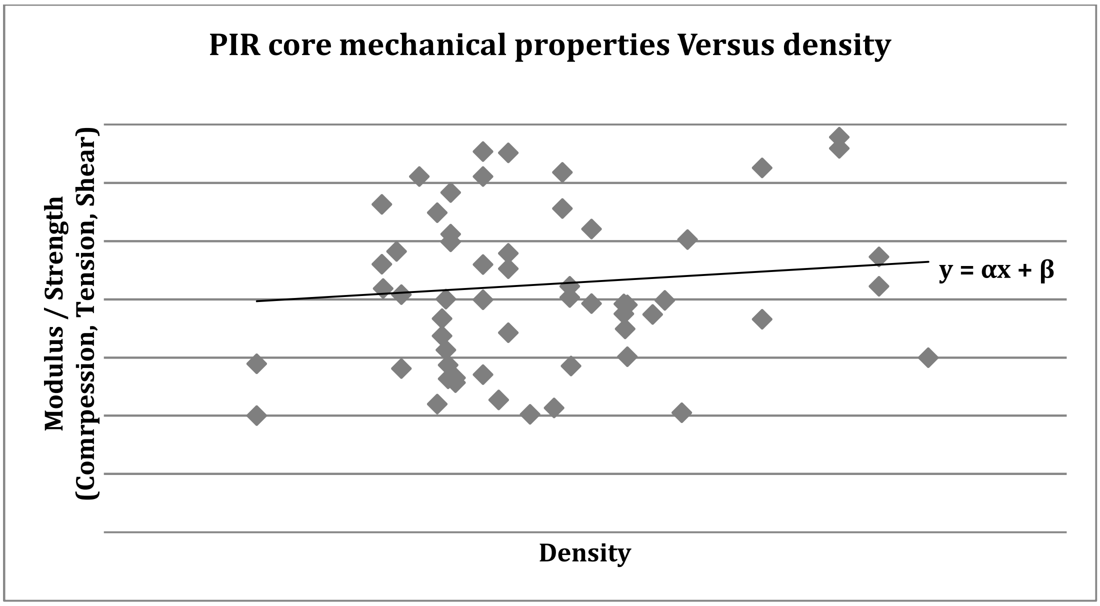


Figure 7.2 Illustration of PIR core mechanical properties versus density plots

The general expressions of the derived mathematical relationships between core mechanical properties and density are shown in Equation 7.1 to Equation 7.6. The full mathematical relationships are shown in Equation E.1 to Equation E.6 in Appendix E.

$$E_{Cc} = \alpha_1 \, \rho_C + \beta_1 \qquad \textbf{Equation 7.1}$$

$$E_{Ct} = \alpha_2 \, \rho_C + \beta_2 \qquad \textbf{Equation 7.2}$$

$$G_C = \alpha_3 \, \rho_C + \beta_3 \qquad \textbf{Equation 7.3}$$

$$f_{Cc} = \alpha_4 \, \rho_C + \beta_4 \qquad \textbf{Equation 7.4}$$

$$f_{Ct} = \alpha_5 \, \rho_C + \beta_5 \qquad \textbf{Equation 7.5}$$

$$f_{Cv} = \alpha_6 \, \rho_C + \beta_6 \qquad \textbf{Equation 7.6}$$

The mechanical properties calculated according to Equation E.1 to Equation E.6 are mean (average) values of the analysed data sets. Notably, Figure E.1 and Figure E.2 show considerable data scattering. BS EN 14509:2013 requires that characteristic values are used in design. The characteristic values derived from tests were calculated according to Equation 7.7:

$$x = \bar{y} - k\sigma_y$$ **Equation 7.7**

Where

x is the characteristic value

$\bar{y}$ is the mean value

σ_y is the standard deviation of the test sample

k is the fractile factor, depending on the number of test specimens

The standard deviation for the sample is as shown in Table E.1. The fractile factor value was according to BS EN 14509:2013.

Use of Equation E.1 to Equation E.6, Equation 7.7 and Table E.1 allows to determine the characteristic values for each mechanical property to be used in design calculations.

7.3.2 Poisson ratio for PIR

The Poisson ratio of the PIR is required in order to estimate the theoretical resistance of the steel sheets when stabilised by the core. Experimental investigations of the Poisson ratio for the PIR-core were undertaken, due to the lack of robust data in the literature and manufacturers' databases. The PIR density was not varied.

The Poisson ratio is defined as inEquation 7.11

$$\nu = \frac{d\varepsilon_{transverse}}{d\varepsilon_{axial}}$$ **Equation 7.8**

for a specimen compressed or stretched in the axial direction, where

$\varepsilon_{transverse}$ is the transverse strain

$d\varepsilon_{axial}$ is the axial strain

Previous tests to determine the Poisson ratio on sandwich panel PUR-cores were conducted by Kurpiela (2013), with the measured values being between 0.20-0.30. PIR has a similar structure to PUR and this reference was used for comparative purposes. It should be noted that the estimation of the compressive resistance of the sheets is not very sensitive to Poisson ratio discrepancies; nevertheless, a reasonable range of values was required for the needs of the study.

The test apparatus was devised such that PIR cubes were subjected to a small compression load (up to 23kg). The compression load was applied as 5kg weight increments on a flat steel plate, to ensure uniform application. The deformations of each plane were recorded with the aid of mechanical transducers. Steel plates ensured that average deformations along the surface were monitored, i.e. local deviations of the PIR surface from flatness were not influencing the measurements. The apparatus is shown in Figure 7.3. The density of each specimen was recorded; however the sample was too small to make observations and conclusions with regards to the influence of density on the Poisson ratio.

Figure 7.3 Test apparatus to determine Poisson ratio for PIR (with and without applied load)

The Poisson ratio was determined based on the ratio of axial and transverse deformation for small load application, using the approximation in Equation 7.12:

$$\nu = \frac{\Delta L_{transverse}}{\Delta L_{axial}} \qquad \textbf{Equation 7.9}$$

Where

ΔL is the differential deflection in the given direction.

The full test results are shown in Table E.4.

The average value of ν=0.09 was adopted to be used in the calculations of the compressive resistance of the sheets in Section 7.4.

7.4 Investigation of compressive resistance of steel sheets

As discussed in Section 7.2, the compressive resistance of the sheets influences the bending resistance of the panel. Hence, an investigation was undertaken to examine the influence of various parameters in the resistance.

For a sandwich panel in bending, lightly profiled steel sheets are subject to axial compression and tension; fully profiled sheets are also subject to bending moment arising from the depth of the profiles (Stamm and Witte, 1974, Davies *et. al.*, 2001). The resistance of the sheets depends on:

- The geometry of the profiles
- The steel strength
- The stiffness of the core, acting as an elastic means to stabilise the steel plates against axial compression.

The load carrying capacity of the steel sheets was investigated with a combination of experimental and analytical programme. The programme was devised to:

- Experimentally examine the compressive resistance of fully and lightly profiled sheets
- Compare the results against the existing design guidance
- Select the current theoretical method which best predicts the experimental performance.

The results of the study were used to examine the reliability of material properties and design guidance on the predicted performance of sandwich panels and incorporate the selected design guidance in the optimisation model in Section 7.6.

7.4.1 Test specimens

The test programme was devised to examine the structural response of commercially available sandwich panels with various sheet geometries in single-span arrangements under structural load and determine their bending capacity, from which the stresses in the sheets can be obtained by calculation. A series of bending tests were conducted on two types of steel-faced sandwich panels with PIR cores and S220 steel grade:

- Type A: One face fully profiled and one face lightly profiled, 120mm deep cores and 31.3mm deep profiles:
    - One type of outer sheet (fully profiled)

   - Two types of liner sheet (lightly profiled)
- Type B: Both faces lightly profiled (outer sheet micro-ribbed) and 120mm deep cores:
   - Four types of liner sheet (lightly profiled)

Different thickness and sheet geometries were used to the available extent, so that sections with different slenderness ratio and stiffness are investigated. The cross-section of the panels and their nominal dimensions are shown in Figure 7.4 and Figure 7.5. The performance of the micro-ribbed sheet for panel Type B was outside the scope of the investigation.

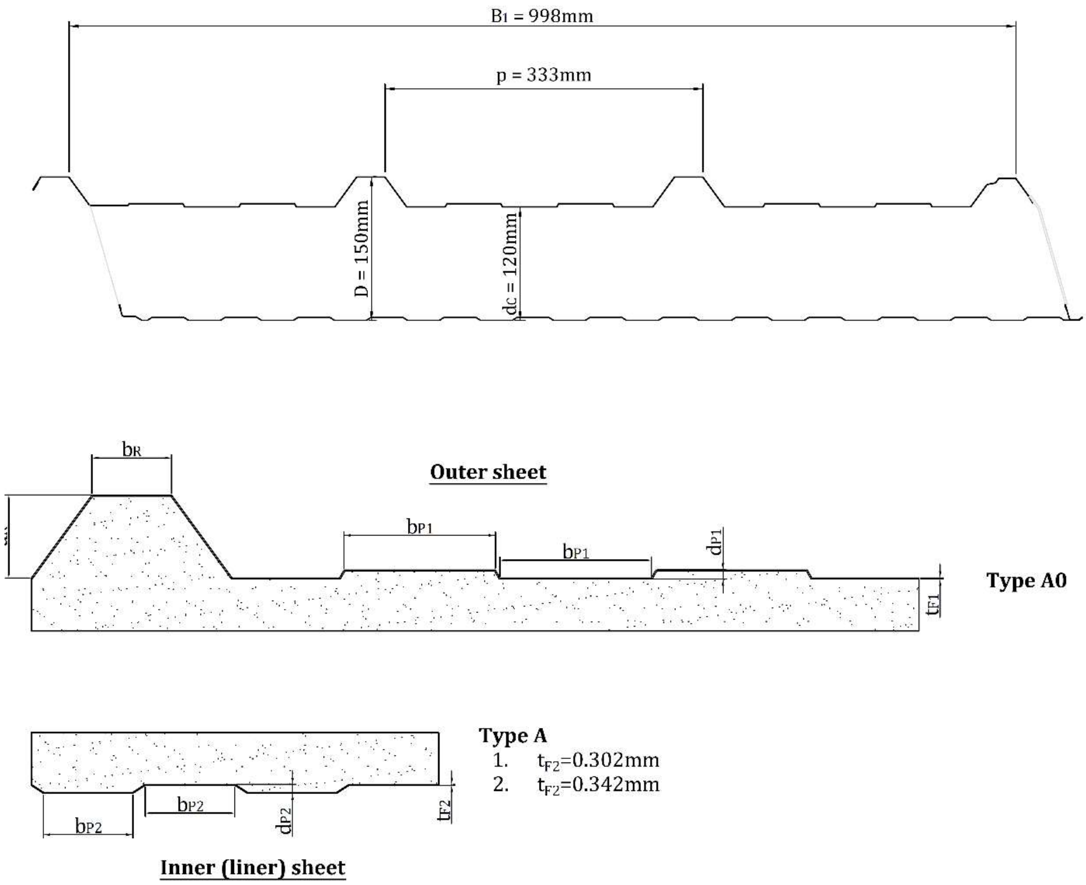
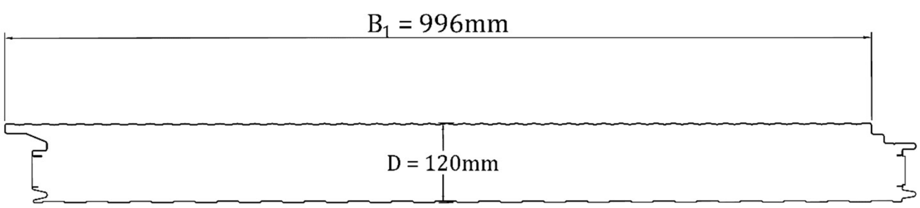


Figure 7.4 Type A panel and sheet types



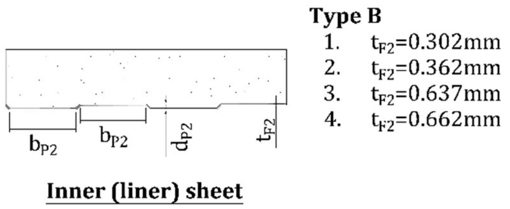


Figure 7.5 Type B panel and sheet types

The geometrical properties of the fully and lightly profiled sheets are shown in Table 7.1 and Table 7.2.

Table 7.1 Geometrical properties for fully profiled sheets (Type A0)

Sheet type	Profile height, d_R (mm)	Crest plate width, b_R (mm)	Rib height, d_{P1} (mm)	Gauge, t_{F1}* (mm)	Flat plate width, b_{P1} (mm)	Area (mm^2)	Moment of inertia (mm^4/m)
A0	31.3	30	2.8	0.442	57	495	51753

**Excluding galvanise*

Table 7.2 Geometrical properties for lightly steel sheets (Type A1-A2, B1-B4)

Sheet type	Rib height, d_{P2} (mm)	Gauge, t_{F2}* (mm)	Flat plate width, b_{P2} (mm)	Slenderness ratio, b/t	Area (mm^2)	Moment of inertia (mm^4/m)
A1	3.0	0.302	33.8	111.9	352	671.84
A2	3.0	0.342	33.8	98.8	310	592.46
B1	1.2	0.362	23.5	77.8	364	118.67
B2	1.2	0.302	23.5	64.9	304	97.71
B3	1.2	0.637	23.5	34.8	642	227
B4	1.2	0.662	23.5	33.6	666	222

**Excluding galvanise*

The material properties of the steel faces and the PIR core were provided by the manufacturer. The mechanical properties were derived from samples taken from panels from the same production run as that of the test specimens and determined according to the test procedures of EN 14509:2013. The measured and nominal material properties are shown in Section E.2 in Appendix E. These data were used throughout the theoretical analysis of the tests specimens.

The measured PIR shear modulus was obtained based on the procedures of BS EN 14509:2013 and concerns the global shear response. However, for the calculation of the compressive resistance of the sheets, the shear modulus at the interface of the core and sheet is required. This may be significantly different to the shear modulus derived for the global response. This is possible to be derived by the compression and tension moduli and the Poisson ratio according to the basic formula shown in Equation 7.10. The results are shown in Table E.4, when using the Poisson ratio derived in Section 7.3.2.

$$G_C = \frac{E_{C,mean}}{2(1+v)}$$ **Equation 7.10**

Where

$$E_{C,mean} = \frac{E_{Cc} + E_{Ct}}{2}$$

7.4.2 Test apparatus

The test apparatus was set according to BS EN 14509:2013 to determine the resistance of the panel in bending and the compressive resistance of the steel sheets. For these tests, a simply supported panel was subjected to four line loads extending across the full width of the panel and applied through hydraulic jacks. A span of 6.0m was selected in order to ensure bending failure (rather than shear or crushing of the core). The applied load was logged, together with deflection measurements at several locations. A schematic of a typical 'simply supported panel with four line loads' test arrangement is shown in Figure 7.6. The test set-up in the laboratory is shown in Figure 7.7 and Figure 7.8 for a fully profiled and a lightly profiled face respectively in compression. A full description of the test apparatus and recording arrangements is given in Section E.3 in Appendix E.

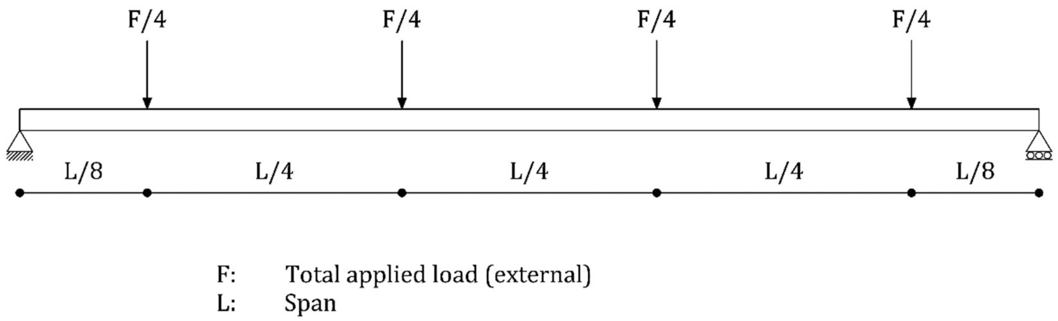


Figure 7.6 Schematic showing a 'simply supported panel with four line loads' arrangement according to BS EN 14509:2013

Figure 7.7 Test arrangement for fully profiled simply supported panel with four line loads

Figure 7.8 Test arrangement for lightly profiled simply supported panel with four line loads

Tests for both face orientations were carried out for panel Type A to investigate both outer and liner sheets in compression. For the panel Type B only the lightly profiled liner sheet was related to the scope of the current research, hence tests only in the orientation causing the liner sheet to be in compression are reported.

The load application for the single-span panel arrangement caused:

- compressive forces in the outer sheet and tensile forces in the liner sheet in the spans
- support reaction forces, compressing the core at the support locations
- shear forces across the panel length with maximum magnitudes close to the supports.

7.4.3 Test results and observations

The test results in terms of failure modes and mean compressive stresses in the sheets are shown in Table E.5. The compressive resistance in the sheets was calculated based on the provisions of BS EN 14509:2013. The full test results showing the total applied load are given in Section E.4 in Appendix E.

Table 7.3 Test results: failure mode and stress at failure

Panel type	**Sheet type (in compression)**	**Failure mode**	**Mean compressive stress in sheets at failure (N/mm^2)**
A	A0	Local buckling	332.7
	A1	Local buckling and flexural wrinkling (mixed mode)	210.3
	A2		200.9
B	B1		181.8
	B2		153.7
	B3		210.3
	B4		165.3

All tests exhibited bending failure associated with failure of the sheet in compression. For panel Type A with the fully profiled sheet (A0) in compression, the response was elastic until local buckling was initiated at the crests of the profile and loss of stiffness was demonstrated. However, this did not lead to global failure because the profiles exhibited ductile post-buckling response. As the load increased, ultimate failure occurred at the lightly profiled troughs of the sheet in a combination of local buckling and flexural wrinkling, causing loss of stability in the sheets and a plastic hinge in the span. For panel Type A and Type B with the lightly profiled sheets in compression, the response was elastic until local buckling was initiated in the flat plates, closely followed by flexural wrinkling and a sudden loss of stability in the sheets, which lead to a plastic hinge in the mid-span. The buckles were more obvious in panel Type A, where the ribs were deeper.

7.4.4 Comparison to theory

The compressive resistances of the steel sheets derived by testing were compared against the available theoretical methods.

For the fully profiled sheets the compressive stress derived by testing was compared against:

- The fully effective capacity of the sheets
- The partially effective capacity according to the design method by Pokharel and Mahendran (2005)

The results are shown in Table 7.4 and demonstrate that the section is nearly fully effective thanks to the stabilising function of the core to the sheet. The design guidance by Pokharel and Mahendran (2005) was found to significantly under-predict the ultimate strength of the sheet in compression. This may be explained by the fact that the guidance focused on high-strength steels which are less ductile compared to normal steels, hence post-bucking resistance can be underestimated.

Table 7.4 Comparison of test results and theory for fully profiled sheets

Sheet type	Test – mean (N/mm^2)	Fully effective section (N/mm^2)	Ratio	Pokharel and Mahendran (2005) (N/mm^2)	Ratio
A0	332.7	357	0.93	203.7	0.61

For the lightly profiled sheets the compressive strength derived by testing was compared against the following theoretical methods:

- Pokharel and Mahendran (2004)
- Davies *et. al* (2001)
- CIB (2000) and
- BS EN 14509:2013.
- Pokharel and Mahendran (2005)

The design method by Pokharel and Mahendran (2005) is for profiles susceptible to local buckling. Since local buckling was observed clearly for the lightly profiled sheets A1 and A2, it was decided that the design method is also included in the comparison.

The results for each sheet type are shown in Figure 7.9. A comparison based on the slenderness ratio and the bending stiffness of the sheets is also shown in Figure 7.10 and Figure 7.11. The theory over test ratios are shown in Table 7.5.

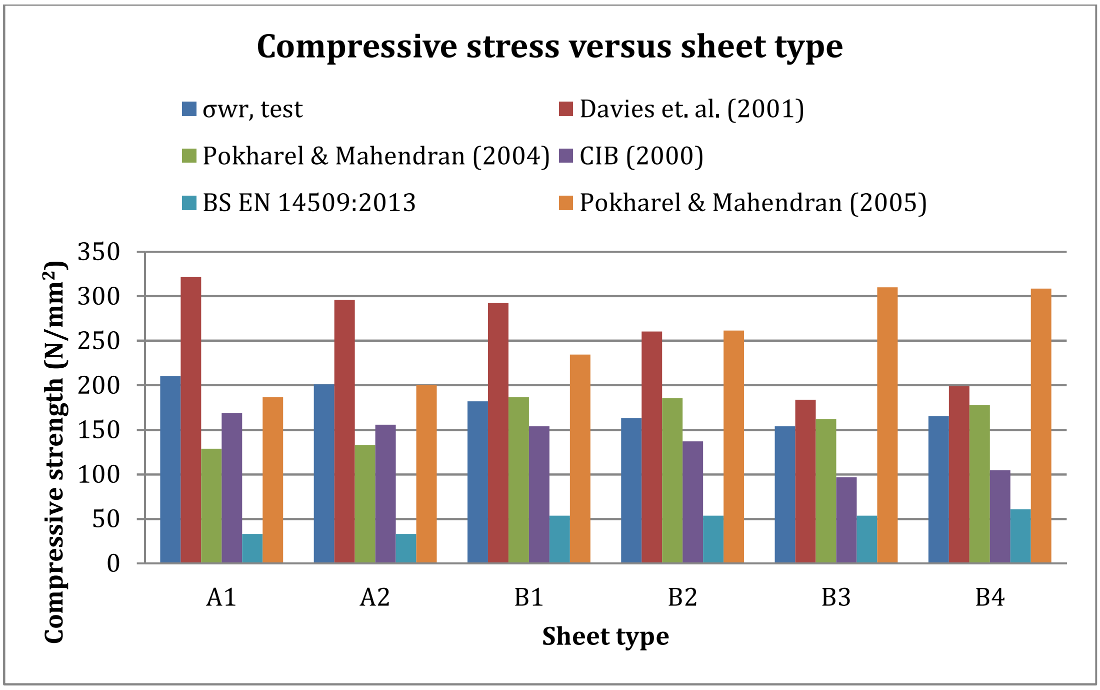


Figure 7.9 Compressive stress versus lightly profiled sheet type: test and theory comparison

Table 7.5 Theory / test ratios (lightly profiled sheets)

Sheet type	**Theory / test ratio**				
	Pokharel & Mahendran (2004)	Davies *et. al.* (2001)	CIB (2000)	BS EN 14509:2013	Pokharel & Mahendran (2005)
A1	0.61	1.53	0.80	0.16	0.89
A2	0.66	1.47	0.77	0.16	1.00
Mean (A1-A2)	*0.64*	*1.50*	*0.79*	*0.16*	*0.94*
B1	1.03	1.61	0.85	0.29	1.28
B2	1.14	1.59	0.84	0.33	1.60
B3	1.05	1.19	0.63	0.35	2.01
B4	1.08	1.20	0.63	0.37	1.86
Mean (B1-B2)	*1.07*	*1.40*	*0.74*	*0.33*	*1.69*
Mean (all)	***0.93***	***1.43***	***0.75***	***0.28***	***1.44***

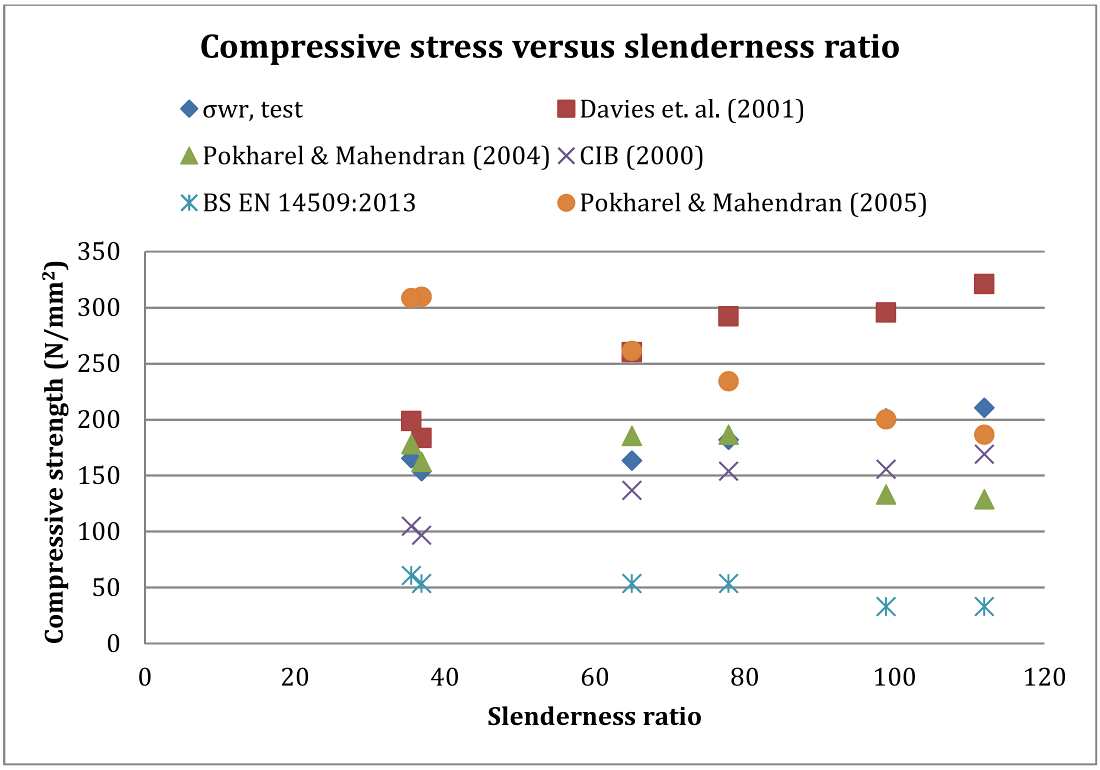


Figure 7.10 Compressive stress versus lightly profiled sheet slenderness ratio: test and theory comparison

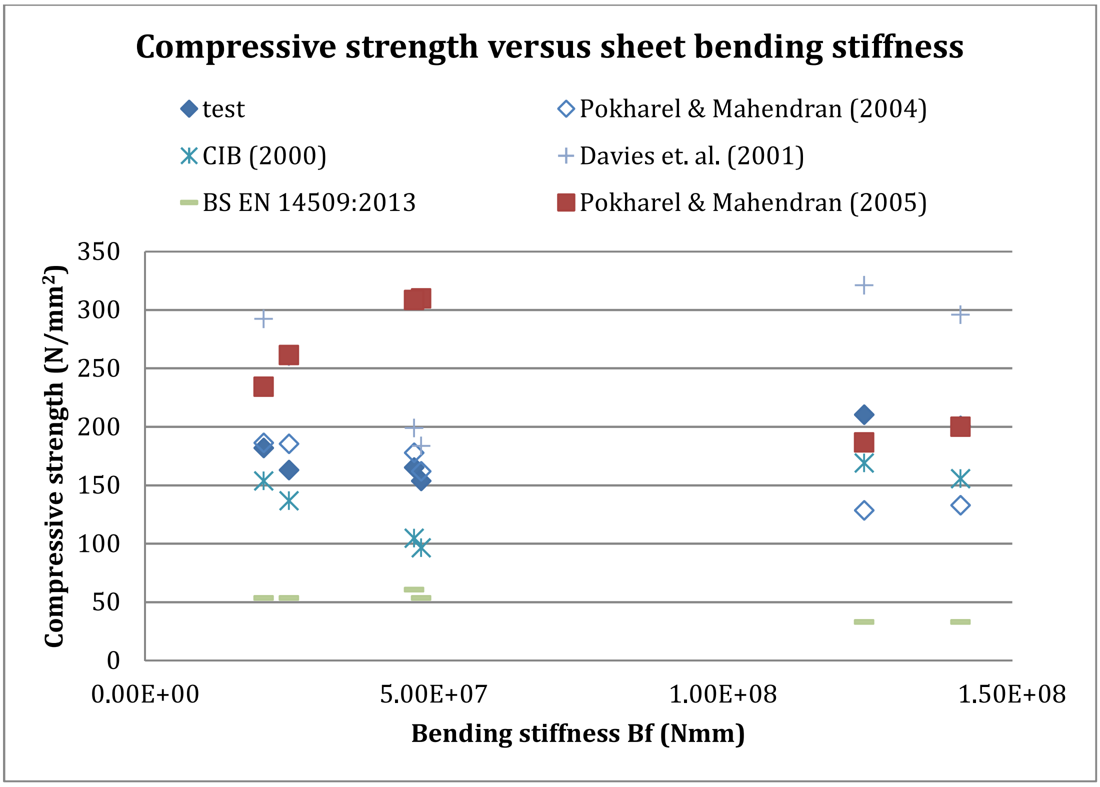


Figure 7.11 Compressive stress versus lightly profiled sheet bending stiffness: test and theory comparison

The results in Figure 7.9 show that the method by Pokharel and Mahendran (2004) and Pokharel and Mahendran (2005) provide the most reliable results for sheet types B1-B4 (those with shallower ribs) and A1-A2 respectively (those with deeper ribs). CIB (2000) was also found to always lead to safe results, while Pokharel and Mahendran (2004) were slightly unconservative. Davies *et. al.* (2001) was found to be very uncoservative, while the simplified formula in BS EN 14509:2013 is extremely conservative.

Similar observations can be made when comparing on the basis of slenderness in Figure 7.10, where Pokharel and Mahendran (2004) was found more reliable for low slenderness ratios and Pokharel and Mahendran (2005) for higher slenderness ratios.

When comparing on the basis of bending stiffness in Figure 7.11, Pokharel and Mahendran (2004) was found to yield very reliable results for small magnitudes, while Pokharel and Mahendran (2004) was found to yield reliable results for higher magnitudes.

7.4.5 Selection of design guidance for further use

The comparison of the results derived by testing show the importance of structural testing to reliably calculate the resistance of the sheets in compression. In Section 7.6 an optimisation model is developed with a large number of combinations of panel specifications. The resistance of the sheets cannot be determined by testing for such a large number of cases, hence the most appropriate design methods with the highest reliability were selected. Improvements in the current design guidance were outside the scope of the present research.

Based on the results of the comparison between theory and testing, the following methods and approaches were selected:

- For fully profiled panels, the steel sheets were considered fully effective. Although 93% effectiveness was demonstrated, it will later be shown that this discrepancy is not critical for long-span applications.
- For lightly profiled panels, the design method by Pokharel and Mahendran (2004) was selected for profiles of low slenderness and bending stiffness and the design method by Pokharel and Mahendran (2005) for profiles of higher slenderness and bending stiffness.

7.5 Influence of material properties reliability on design performance

For the reference sandwich panel (Appendix A) with nominal mechanical and geometrical properties and incorporating the test results for the compressive resistance of the sheets, the resistance utilisations in pressure and uplift for the maximum allowable loads at 6.0m single-span are shown in Table 7.6.

Table 7.6 Resistance utilisations for reference fully profiled panel at 6.6m single-span

	Pressure	**Uplift**
Characteristic load	0.60kN/m^2	-0.88kN/m^2
Bending – outer	0.67	0.75
Bending – inner	0.65	1.00
Shear stress	0.31	0.33
Support stress	0.43	0.44
Deflection	1.00	0.84

It is shown that for long span applications, stiffness governs in pressure and the compressive resistance of the inner (liner) sheet in uplift. Also, the maximum allowable uplift load for the panel is higher than that required for the buildings in the present study

(0.66kN/m^2 for medium size and 0.78kN/m^2 for large size – see Appendix B). Hence, the focus is on the pressure load case which is typically governed by the imposed load for roofs accessible for maintenance (0.6kN/m^2 - see Appendix B).

BS EN 14509:2013 requires that a correction factor is applied to the compression resistance derived by testing for fully profiled sheets. A large deviation between observed and nominal yield strength values, even if the observed is higher, requires a reduction to the design compressive strength. This approach is not relevant to the lightly profiled sheets, for which yield strength is practically irrelevant.

For the compression resistances of the fully profiled sheets and their nominal and observed yield strength, the adjusted strengths which would be allowed by the code for design are shown in Table 7.7.

Table 7.7 Adjusted design values according to BS EN 14509:2013 based on observed and nominal yield strength for fully profiled sheets

Strength	Value		
$f_{y,obs}$	357N/mm^2		
$f_{y,nom}$	220N/mm^2	280N/mm^2	350N/mm^2
R_{adj}	243N/mm^2	274N/mm^2	307N/mm^2

This shows that an increase in the design resistance of the fully profiled sheets is possible simply by increasing the reliability of the nominal yield strength. However, the compressive resistant of the fully profiled sheet does not govern the design, any further improvements in the material properties reliability would not allow increase in the allowable maximum applied loads.

The stiffness of the panel is, however, largely reliant on the shear stiffness of the core. As shown in Figure E.1, the difference between the mean and minimum shear stiffness values is considerable. This is captured by the standard deviation which penalises the nominal value of the property. Consequently, further improvements in the design performance of the panel can be achieved by increasing the reliability of the core material properties. It was not feasible to identify the extent of the improvements required by statistical analysis due to the wide scatter of the mechanical properties of the PIR. For illustrative purposes only, however, an example is given in Table 7.8 for the maximum allowable load increase for 10% increase of the shear modulus (from the nominal, as shown in Appendix A) and the mean values. The results show that some improvement in the maximum allowable load is feasible, however this is very small.

Table 7.8 Influence of shear modulus reliability on resisting load (6.6m single-span)

	Max allowable pressure	**Max allowable uplift**
Shear modulus: +10%	0.61kN/m^2	-0.88kN/m^2
Shear modulus: mean	0.65kN/m^2	-0.88kN/m^2

7.6 Sandwich panel optimisation for long span requirements

7.6.1 Mathematical basis

The general optimisation problem with multiple objectives can be defined as in Equation 7.11 (Marler and Arora, 2004):

$$\text{minimise } f(x) = [f_1(x), f_2(x), \dots f_k(x)]^T \qquad \textbf{Equation 7.11}$$

Where k is the number of objectives and x is the vector including all the problem variables.

The optimisation problem is subject to equality and inequality constraints in Equation 7.12 and Equation 7.13 respectively (Marler and Arora, 2004):

$$g_j(x) \leq 0 \; with \; j = 1,2, \dots, m \qquad \textbf{Equation 7.12}$$

$$h_l(x) = 0 \; with \; l = 1,2, \dots, e \qquad \textbf{Equation 7.13}$$

Where m is the number of inequality constraints and e the number of equality constraints.

For a multi-objective optimisation problem, there is no single optimal solution and the problem relies on identifying a set of solutions which fit within a definition of optimum (Marler and Arora, 2004). For such problems, the concept of Pareto-optimality is frequently used in engineering and has recently been successfully used for sandwich panel optimisation (Kurpiela, 2013, Kurpiela and Lange, 2013).

7.6.2 Problem formulation

The problem was defined as maximising the resisting load of the panel and minimising the embodied carbon for the chosen span distances. The term ‘resisting’ load refers to the characteristic load applied to the panel prior to any load factors applied. The simultaneous consideration of resisting load and embodied carbon forms a problem with two competing objectives. When the required load to be resisted is known for the given span distance, a single optimal solution can be found. The problem may be described in the form show in Equation 7.14, being subject to inequality and equality constraints.

$$\text{minimise } f(x) = [f_1(x), f_2(x)^T] \qquad \textbf{Equation 7.14}$$

Where

$f_1(x) = 1/q$ is the strength objective and

$f_2(x) = E_{CO_2}$ is the embodied carbon objective.

The objective function $f_1(x) = 1/q$ represents the inverted value of q, with q being the maximum characteristic load arising from the variable actions and being resisted by the panels in either pressure or suction.

The objective function $f_2(x) = E_{CO2}$ expresses the embodied carbon of the sandwich panel, using Equation 7.15 (see Section 7.6.2.1).

The general plot of the Pareto-optimal solutions set with the two competing objectives functions $f_1(x)$ and $f_2(x)$ is shown in Figure 7.12 (a), where solutions are presented in the form of the results for each objective $f_1(x)$ and $f_2(x)$. In order to facilitate easier inspection, the objectives can be presented in the form of $1/f_1(x)$ and $f_2(x)$, as shown in Figure 7.12(b), representing the relationship between strength and embodied carbon for each solution.

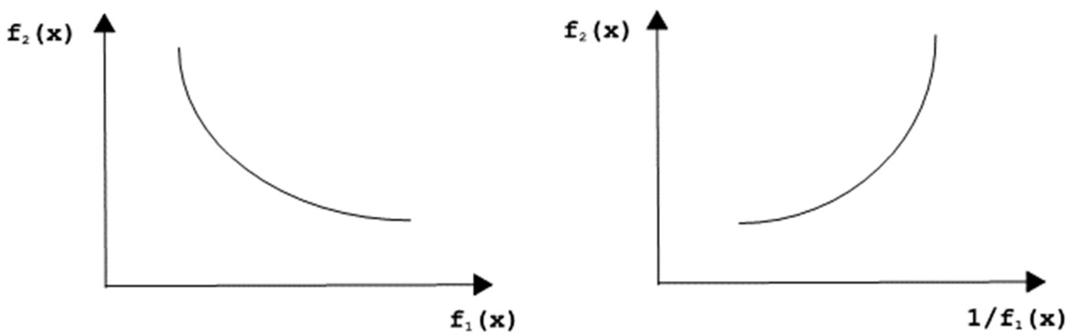


Figure 7.12 General plot of Pareto-front optimisation solutions

7.6.2.1 Allowable variable characteristic load

The maximum allowable variable characteristic load was calculated according to BS EN 14509:2013. The structural arrangements for the panels were modelled as single-spans at 8m distance with pinned-end conditions and uniformly distributed load (UDL) applied across their length, as shown in Figure 7.13.

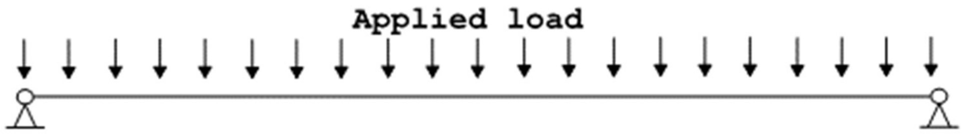


Figure 7.13 Structural and load arrangement for optimisation model

Allowance was made for the self-weight of the panels. The applied load due to variable actions (imposed, wind, snow) was correlated to the strength objective function $f_1(x)$.

Furthermore, load due to temperature effects was included, based on the colour group and the guidance in BS EN 14509:2013. For the requirements of the present study and further to consultation with manufacturers, light colours were considered for the outer face of the panels, being representative of typical roof applications. The temperatures used for the inner and outer sheets in summer and winter are shown in Table 7.9.

Table 7.9 Temperature loads on sandwich panels

Temperature on outside sheet T_1		**Temperature on inner sheet T_2**		**Temperature difference ΔT**	
Summer	Winter	Summer	Winter	Summer	Winter
+65°C Light colour	-10°C For the UK	25°C	20°C	-40°C	30°C

The load factors and combinations were according to BS EN 1990:2002 and BS EN 14509:2013, as shown in Appendix B. It is typical manufacturer's practice to provide accredited load-span tables, which include the following load combinations:

- Permanent + Imposed (ULS, SLS)
- Permanent + Wind uplift (ULS, SLS)
- Permanent + Imposed + Temperature (SLS)

The maximum allowable loads calculated in this exercise are based on the above combinations.

The load factors used for Ultimate Limit States are summarised as follows:

- Load factor for permanent load, γ_G: 1.35 (ULS)
- Load factor for imposed load / wind, γ_Q: 1.50 (ULS)

For Serviceability Limit States the load factors were set equal to 1.0 for all the variables. For the temperature combination, the following load factors were applied:

- Load factor for permanent load, γ_G: 1.00
- Load factor for imposed load / wind, γ_Q: 0.75
- Load factor for temperature load, γ_G: 0.60

7.6.2.2 Embodied carbon quantification

The embodied carbon of the sandwich panel assemblies was calculated using the database of the University of Bath's Inventory of Carbon and Energy (ICE) (Hammond and Jones, 2008). The particular database adopts 'cradle-to-gate' system boundaries, i.e. transportation of materials to site, construction processes and end-of-life options are excluded, which was considered appropriate for the particular study for the following reasons:

- The stages included in the 'cradle-to-gate' approach yield a high level of certainty and remain unchangeable for the manufactured product regardless of its treatment once it leaves the factory gate. On the other hand, transportation, construction and end-of-life stages vary significantly, depending on the project.
- Transportation of materials to and from the manufacturing, construction and waste sites is highly project-specific and depends on the site's location which may vary significantly. Overall, it was judged that robust assumptions for the transport effects could not be made within the generic context of the exercise.
- There is no standard industry approach for the end-of-life options and there is lack of a common database for impacts of the steel and PIR foams.

The ICE comprises a single database and transparent methodology for the sandwich panel materials as well as the whole range of construction materials included in the present study and discussed in more detail in Chapter 8. Hence, consistency and a single reference source for the materials used for the production of the sandwich panels and the materials for the whole building within the study is guaranteed.

It is important to highlight that the assumed energy used in manufacture and the consequent embodied carbon emissions of materials and products could change substantially in the future as the energy grid efficiency and the energy mix sources change. The embodied carbon of materials is formed by two sources: fossil fuel inputs and released greenhouse gases (Hammond and Jones, 2008). As the electricity grid emissions in the UK, as well as in rest of Europe, reduce with time and as the energy source mix is already in the trajectory of shift towards less energy produced by fossil fuels and more by renewables and other non-fossil fuel sources, it is likely that the embodied carbon of materials included in Bath's ICE database is subject to major changes in the future. The sensitivity of the resulting embodied carbon of materials and products to energy grid efficiency and source mix changes is important to be evaluated in future studies, whilst modifications to the embodied carbon databases across different locations due to differences in grid and mixes need to be progressed in the future.

Further to consultation with panel manufacturers, it is deemed that manufacturing itself (incl. sheet profiling, foam spraying, panel stacking etc.) has only as small contribution in the overall embodied carbon of a panel, where the greatest contribution comes from the materials themselves. Furthermore, there is no robust database for energy and carbon required during manufacturing process. Hence it was judged that manufacturing can be excluded from the present comparative study with confidence.

The following embodied carbon emissions according to Hammond and Jones's (2008) ICE were used in the analysis:

- $k_{PIR} = 4.26$kgCO_2/kg
- $k_{Steel} = 1.54$kgCO_2/kg

Hence, the objective function $f_2(x)$for embodied carbon of sandwich panels is expressed as:

$$f_2(x) = k_{PIR}W_{PIR} + k_{Steel}W_{Steel}$$ **Equation 7.15**

Where $f_2(x)$is expressed in kgCO_2e/m^2, using the carbon coefficients of PIR (k_{PIR}) and galvanised steel sheeting (k_{Steel}) and W_{PIR} and W_{Steel} referring to the weight of PIR and steel respectively.

7.6.3 Optimisation variables

The variable vector x includes all the optimisation variables, i.e. the geometrical and mechanical properties which influence the strength and stiffness behaviour of a sandwich panel with one face fully profiled and one face lightly profiled. The vector may be described as in Equation 7.16:

$$x = [d_C, p, d_R, b_R, \varphi_R, b_{p11}, b_{p12}, b_{p13}, b_{p14}, d_{p1}, \varphi_{p1}, b_{21}, b_{22}, \varphi_{p2}, d_{p2}, G_C, E_{Cc}, E_{Ct}, f_{Cv}, f_{Cc}, f_{Ct}, E_F, f_{y1}, f_{y2}, t_{F1}, t_{F2}] \quad \textbf{Equation 7.16}$$

An illustration with the definition of the variables is shown in Figure 7.14.

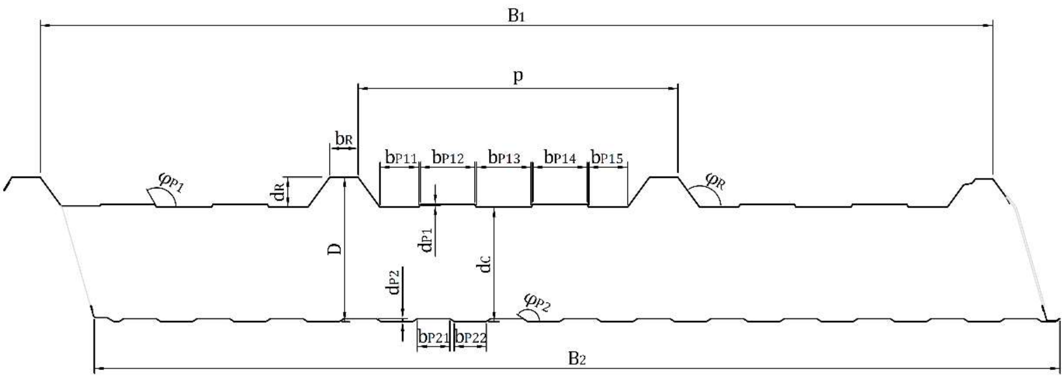


Figure 7.14 Optimisation variables of fully profiled sandwich panel

Each of the core mechanical properties is a function of the core density, as established in Section 7.3.1:

$$E_{Cc} = f_1(\rho_C)$$

$$E_{Ct} = f_2(\rho_C)$$

$$G_C = f_3(\rho_C)$$

$$f_{Cc} = f_4(\rho_C)$$

$$f_{Ct} = f_5(\rho_C)$$

$$f_{Cv} = f_6(\rho_C)$$

The mechanical properties of steel and the geometrical properties of both steel and core are independent values.

The following variables were kept constant as for the standard product:

- p
- φ_R
- d_{p1}
- φ_{p1}
- d_{p2}
- φ_{p2}

The full set of these values is given in Table E.6 in Appendix E. A brief analysis identified that there was little variation for the compressive resistance of the sheets when d_{p1} and d_{p2} were varied.

The outer sheet's troughs was set to comprise four profiles (as for the current reference panel) of equal width which is dependent on the full profile's pitch and width and the trough's profile pitch and depth as per below:

$$d_{p1} = d_{p2} = d_{p3} = d_{p4} = p - b_R - \frac{2\,d_R}{tan\varphi_R} - \frac{8\,d_{p1}}{tan\varphi_{p1}}$$

The width of the liner sheet's troughs and crests was chosen to be the same, i.e.:

$$b_{21} = b_{22} = b_2$$

Furthermore, the yield strength of steel was chosen to be the same for both faces as in standard practice. Hence:

$$f_{y1} = f_{y2} = f_y$$

Since relationships between the core density and its mechanical properties were earlier established, the variable vector can be reduced to Equation 7.17:

$$x = [d_C, d_R, b_2, \rho_C, f_y, t_{F1}, t_{F2}] \qquad \textbf{Equation 7.17}$$

7.6.4 Constraints

Two set of constraints were applied. The first set considered behavioural constraints, adhering to the requirements at Ultimate Limits States (ULS) and Serviceability Limit States (SLS). The second set of constraints included the boundary conditions associated with manufacturing.

7.6.4.1 Behavioural constraints (strength, stability, stiffness)

The constraints were associated with the design checks in terms of strength, stability and stiffness according to BS EN 14509:2013. For single-span, simply supported panels, the behavioural constraints are described by Equation 7.18 to Equation 7.22:

$$\frac{\sigma_{F1}}{f_{F1}} - 1 \leq 0 \qquad \textbf{Equation 7.18}$$

$$\frac{\sigma_{F2}}{f_{F2}} - 1 \leq 0 \qquad \textbf{Equation 7.19}$$

$$\frac{\tau_C}{f_{Cv}} - 1 \leq 0 \qquad \textbf{Equation 7.20}$$

$$\frac{\sigma_{Cc}}{f_{Cc}} - 1 \leq 0 \qquad \textbf{Equation 7.21}$$

$$\frac{w_{max}}{w_{SLS}} - 1 \leq 0 \qquad \textbf{Equation 7.22}$$

Where:

Equation 7.18 and Equation 7.19 refer to strength and stability checks (ULS) of the steel sheeting against local buckling and flexural wrinkling in compression, or yielding in tension, i.e. $f_{Fi} = \begin{cases} f_{yi} \; for\ sheet\ in\ tension \\ \sigma_{wi} \; for\ sheet\ in\ compression \end{cases}$

Equation 7.20 refers to strength checks for the core against shear failure (ULS).

Equation 7.21 refers to strength checks for the core against compression failure. Although this failure is considered at ULS, it does not lead to ultimate failure of the panel. The support width for this failure check was considered constant at 100mm.

Equation 7.22 refers to deflection check for SLS, where $w_{SLS} = Span/200$ for roofs.

The notation has been explained in Section 7.2.

The resistance of the fasteners in tension, pull-out or pull-through was not included in the model, as an appropriate number of fasteners to resist the wind suction load can be easily specified. For single-span arrangements, the number of fasteners does not influence the resistance of the sheets, unlike the case for continuous arrangements (Davies *et. al.*, 2001, BS EN 14509:2013).

The panel resistance corresponding to each behavioural constraint was calculated according to BS EN 14509:2013 and the selected design methods in Section 7.4 for the resistance of the fully and lightly profiled sheets. Since the lightly profiled liner sheet was largely similar to the one tested, the design method by Pokharel and Mahendran (2005) was included in the model.

Material partial safety factors γ_M for steel sheets were nominal values according to BS EN 14509:2013. Those for the cores were according to manufacturers' data for the reference panel, possessing consistent properties. The material partial safety factors used for the current exercise are summarised in Table 7.10.

Table 7.10 Material partial safety factors (γ_M)

Resistance type	Ultimate Limit States	Serviceability Limit States
Yielding of steel sheet	1.10	1.00
Wrinkling* of steel sheet	1.25	1.10
Shear failure of core	1.25	1.00
Crushing failure of core	1.25	1.00
Failure of fastener	1.33	1.00

**Compressive failure*

7.6.4.2 Production and manufacturing constraints

The limits of each variable were also constraints to the model. The following property ranges were adopted for the study:

$$100mm \leq d_C \leq 150mm$$

$$31.3mm \leq d_R \leq 61.3mm$$

$$23.8mm \leq b_2 \leq 43.8mm$$

$$0.45 \leq t_{F1} \leq 0.75mm$$

$$0.35 \leq t_{F2} \leq 0.65mm$$

$$f_y = 350N/mm^2$$

These concerned production and manufacturing limitations and were based on consultation with panel manufacturers. In specific:

- The panel depth limitation is associated with thermal stresses imposed by the high foam temperature at application and requirements for time to cool. Deeper PIR insulations would require longer cooling times after the insulation is laid on the sheets. Empirical evidence suggests that the top and bottom of the insulation cool down faster than the middle. For cores in excess of 150mm this often introduces thermal stresses at production which cause the panel to bend. Further investigation on these thermal effects would be required; however this is outside the scope of the current research.
- It is desirable that the full profile depth is limited in order to allow for full penetration of the foam into the cavities and create full bond with the steel sheeting. Deep profiles in excess of 60mm would not be desirable since they may

lead to inhomogeneous foam distribution within the cavities. This would be likely to reduce the effectiveness of the foam's stabilising function to the thin steel plates and consequently reduce their compressive resistance.

- It is also perceived that reducing the thickness from the current magnitudes would not be advantageous, particularly for the inner sheeting. This is because tight fastening on thinner sheets would be likely to create large indentation of the sandwich sheet around the fastener seal, which is against good construction detailing and should be avoided.

The sheet gauge was limited to 0.75mm for the outer faces and 0.65mm for the liner faces. Although in theory these values could be increased further, it was judged that a sensible limitation should be applied, corresponding approximately to the lower steel gauge values used for decking products.

Finally, an observed and nominal yield strength of 350N/mm^2 was adopted. Yield strength in excess of 350N/mm^2 is already demonstrated by current high quality steel coils with a nominal S220 steel grade. If nominal 350N/mm^2 yield strength could be guaranteed, the reliability would be increased and that would reduce the statistical penalty imposed by BS EN 14509:2013 on the design compressive resistance of the full profiles. It is nevertheless worth recalling that the yield strength showed little influence on the performance of long single-span panels, where the design is governed by the control of deflections, for which yield strength has no influence.

7.6.5 Variable ranges and increments

The range and increments for each variable are summarised in Table 7.11. The increment magnitudes were selected to be integer numbers and such that they limit the size of the problem while sensitive enough to derive conclusions.

Table 7.11 Variable ranges and increments

Variable	Range	Increment
d_C	100mm – 150mm	10mm
d_R	31.3mm – 61.3mm	10mm
b_2	23.8mm – 33.8mm	10mm
ρ_c	38kg/m^3 – 44kg/m^3	2kg/m^3
f_y	350N/mm^2	N/A
t_{F1}	0.45mm – 0.75mm	0.10mm
t_{F2}	0.35mm – 0.65mm	0.10mm

7.6.6 Optimisation process

For the selected span distance, optimisation constraints and each combination of variables for the defined range and increments, a calculation was performed to determine:

- the maximum characteristic imposed and wind uplift load magnitudes, according to BS EN 14509:2013 for the load combinations according to Section 7.6.2.1.
- the resulting embodied carbon, according to Section 7.6.2.2.

Each solution was plotted as a data point corresponding to the load – embodied carbon combination, i.e. $f_2(x) = f(f_1(x))$. The Pareto-front was then formed as the lower bound of the series of data points. For the required characteristic load and the formed Pareto-front curve, the Pareto-optimal solution (combination of variables) was then identified graphically corresponding to the minimum embodied carbon magnitude. The results are shown and discussed in Section 7.6.7.

7.6.7 Results

The results of the optimisation study are presented in Figure 7.15 and Figure 7.16. The graph presents the embodied carbon for variable characteristic load acting on the panels in pressure and suction for the defined load combinations. The data points on the Pareto-front are plotted in green, depicting the set of solutions which are optimal in terms of embodied carbon for each magnitude of characteristic load action on the panel. The non-optimal solutions are also shown on the plots as blue and red for the pressure and uplift cases respectively.

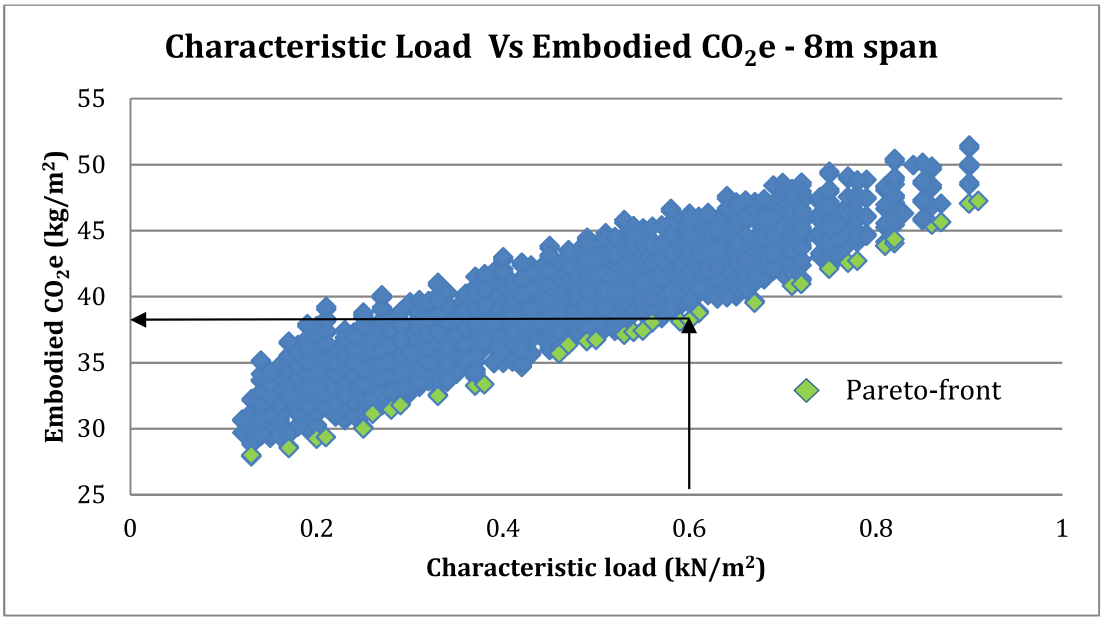


Figure 7.15 Solutions for 8m single-span sandwich panels (pressure)

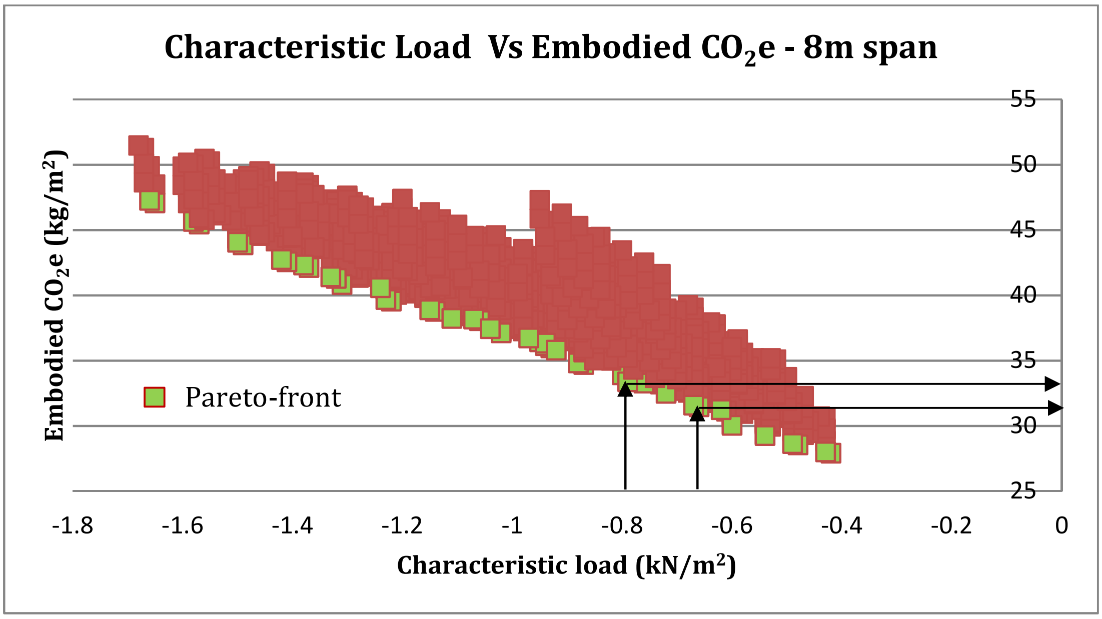


Figure 7.16 Solutions for 8m single-span sandwich panels (uplift)

For the variable load magnitudes acting on the building, the Pareto-optimal solutions and their associated embodied carbon were identified. The following load magnitudes are required to be resisted (Appendix B):

- Imposed load (pressure): 0.6kN/m2
- Wind load (uplift) for the medium building size: 0.66kN/m^2
- Wind load (uplift) for the large building size: 0.78kN/m^2

The optimal solution points are shown on Figure 7.15 and Figure 7.16 with the aid of arrows. The variable values corresponding to each solution in terms of specifications are presented in Section 7.6.9. Prior to presenting those, a sensitivity study was undertaken to examine the extent of influence of each variable and how each variable contributes to the development of the optimised solution. This is discussed in Section 7.6.8

7.6.8 Observations and discussion

The extent of influence of each variable on the structural performance was investigated qualitatively through graphical inspection. The contribution of each variable to the development of the optimised solution was also examined. The solutions were distinguished per variable and the data sets were plotted for each of the variables' values.

Figure 7.17 and Figure 7.18 show the distribution of the solutions based on the core density. The graph clearly illustrates that as the density increases, the increase in the allowable characteristic load is minimal, while the embodied carbon is obviously increasing. The Pareto-front comprises solutions with only the lower core density. This may be explained by the fact that the increase of the shear, compression and tension moduli with the increase in the core density is fairly modest, as observed in Figure E.1. As the performance of the panel in pressure is governed by stiffness, which is heavily influenced by the shear modulus, and the in suction by the compression resistance of the liner sheet, which is considerably influenced by a combination of shear, compression and tension moduli, the modest improvement of the mechanical properties of the core with the increase in core density leads to minimal benefit in terms of allowable characteristic load. Consequently, it is demonstrated that regardless of the load magnitude required to be resisted, the optimised sandwich panel would require the lowest PIR core density (38kg/m^3). This is an important finding considering the very high emission rate of PIR per unit mass.

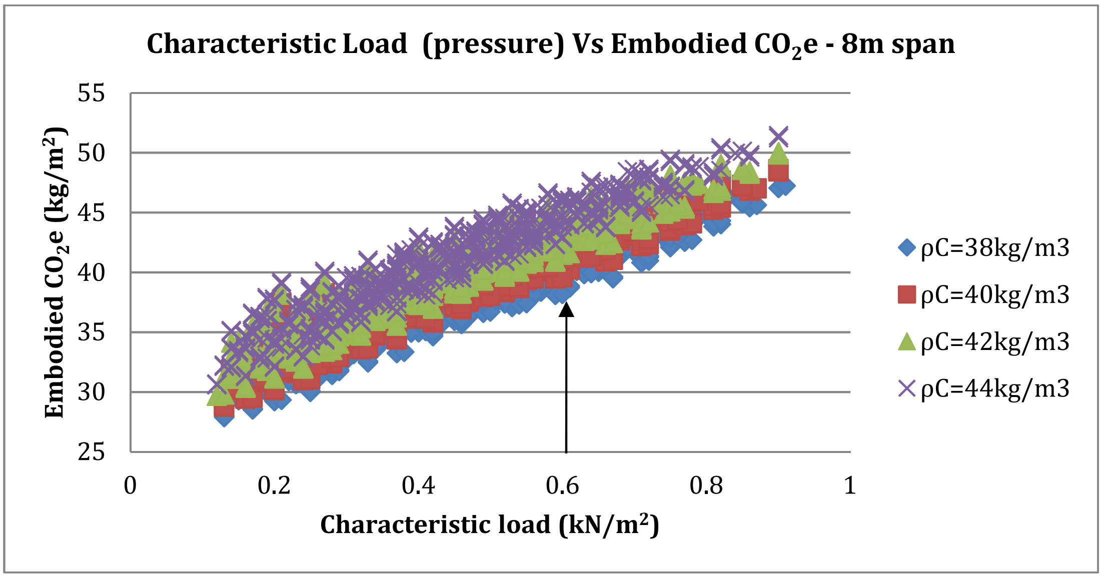


Figure 7.17 Influence of core density on maximum applied load and embodied carbon (pressure)

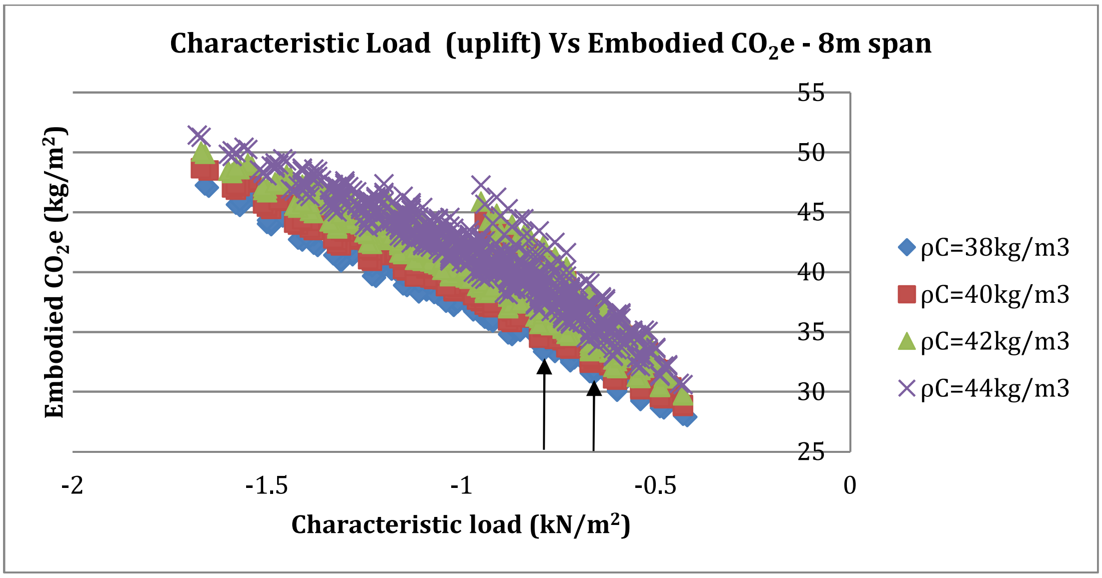


Figure 7.18 Influence of core density on maximum applied load and embodied carbon (suction)

Figure 7.19 and Figure 7.20 show the distribution of solutions based on the depth of the core. The graphs clearly illustrate that as the depth increases, so do the allowable characteristic load and the embodied carbon, in an almost proportionate manner. All the variable magnitudes can be found on the Pareto-front, demonstrating that there is no clear optimum value for the depth of the core. Ultimately, the selected core depth would depend on the load magnitude which is required to be resisted. For the required maximum allowable load, the graphs illustrate that the required core depth would be 135mm for pressure and 100mm or 120mm for uplift for the medium and large building sizes respectively.

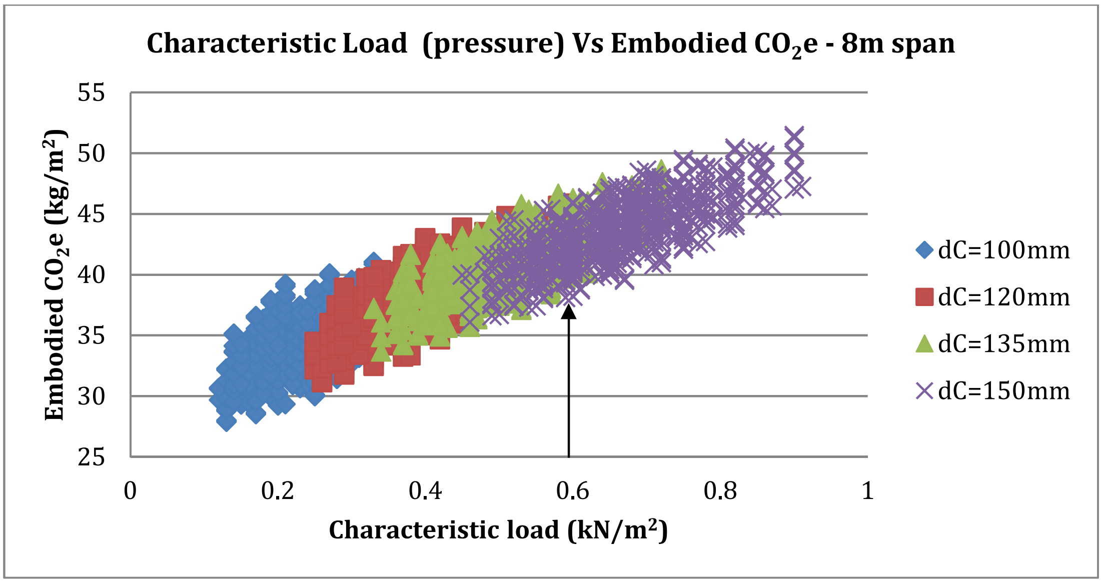


Figure 7.19 Influence of core depth on maximum applied load and embodied carbon (pressure)

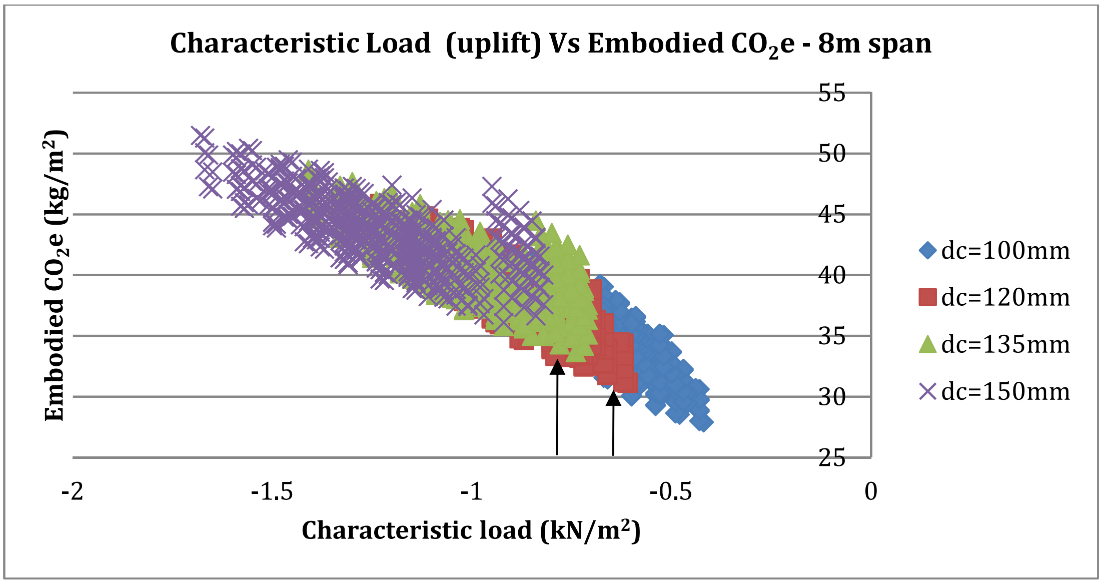


Figure 7.20 Influence of core depth on maximum applied load and embodied carbon (suction)

Figure 7.21 and Figure 7.22 show the distribution of solutions based on the depth of the full outer profile. Similar to the core depth case, the graphs clearly illustrate that as the depth of the profile increases, so do the allowable characteristic load and the embodied carbon, in an almost proportionate manner. The rate of increase appears, however, lesser than for the core depth, i.e. between the two variables, the core depth remains more influential. All the variable magnitudes can be found on the Pareto-front. Notably, the benefit of the deeper full profile depth is more obvious for the pressure case, as a stiffer outer sheet contributes to higher bending stiffness, which governs the design of the panel in pressure. In addition, when the panel is in uplift, the stiffer outer profile attracts higher loads and partially relieves the inner sheet in compression. For the required maximum allowable load, the graphs illustrate that the required depth of the full outer sheet profile depth would be 61.3mm for both pressure and uplift. For the uplift case, the structural benefit of the 61.3mm depth compared to the 51.3mm and 41.3mm depths at the given loads is just marginal.

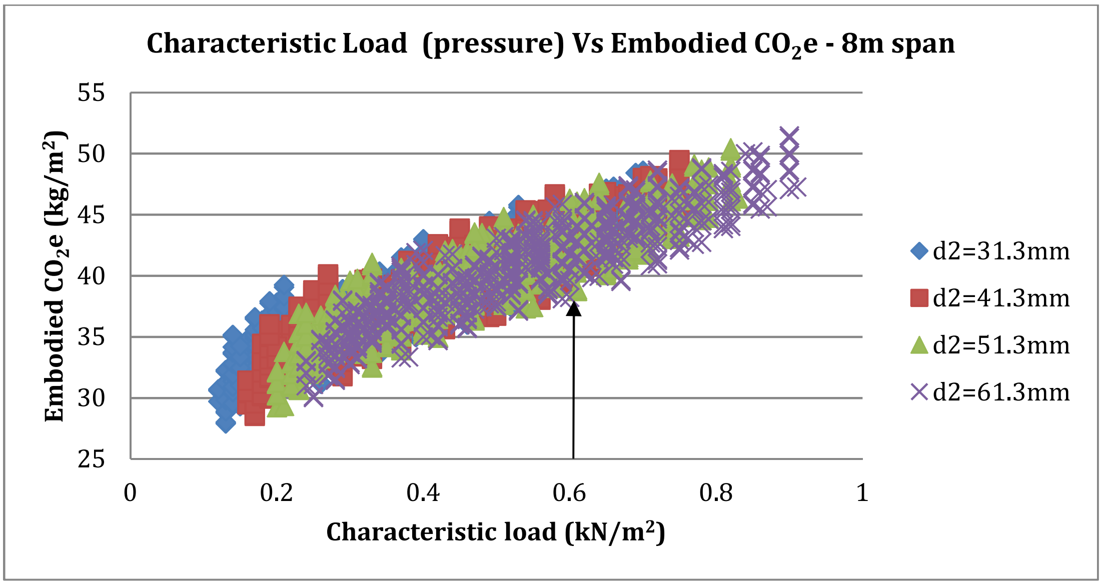


Figure 7.21 Influence of full outer profile depth on maximum applied load and embodied carbon (pressure)

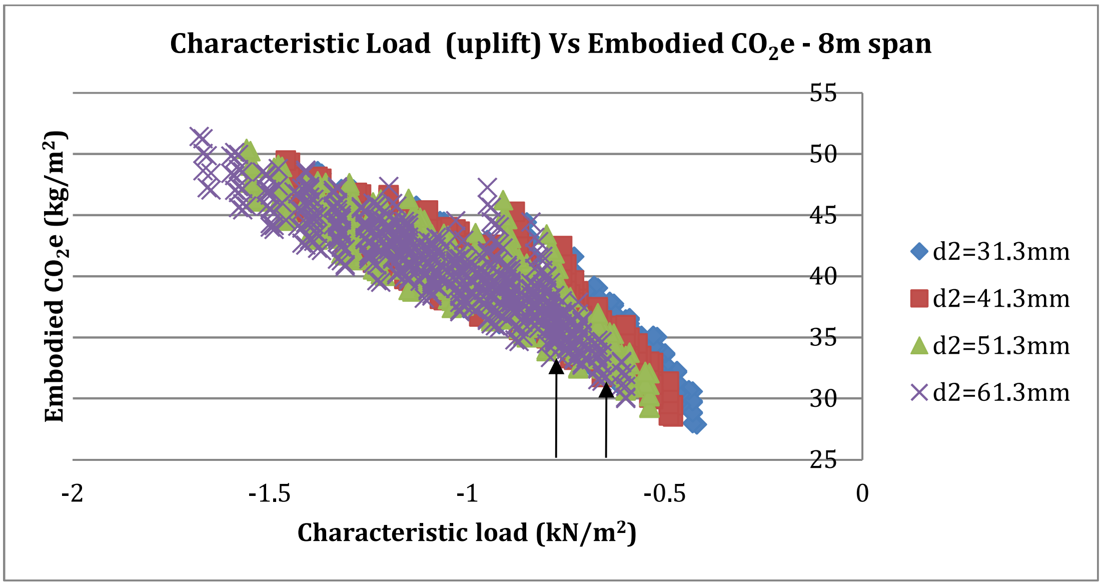


Figure 7.22 Influence of full outer profile depth on maximum applied load and embodied carbon (suction)

Figure 7.23 and Figure 7.24 show the distribution of solutions based on the thickness of the outer sheet. The graphs show that as the thickness increases, there is a small increase in the allowable characteristic load and a clear increase in the embodied carbon. The pattern of increase in the characteristic load is similar in both pressure and suction. Increased thickness of the outer sheet increases the stiffness of the outer sheet and the panel, consequently improving the deflection control, which governs the response in pressure. In addition, as the stiffness of the outer sheet increases, it attracts higher load and this has a relieving effect to the forces distributed to the liner sheet for suction. Nevertheless, the improvement arising from the increased steel gauge of the outer sheet is little compared to that of the profile height increase, while the increase in embodied carbon is obvious. The Pareto-front for both pressure and suction comprises the lowest values of the variables, with the exception of characteristic loads at the high end of the curve, where a shift towards thicker outer sheet is noticed. Consequently, it is demonstrated that for load magnitude required to be resisted in either pressure or uplift, the thinner outer sheet (0.44mm) would be optimal.

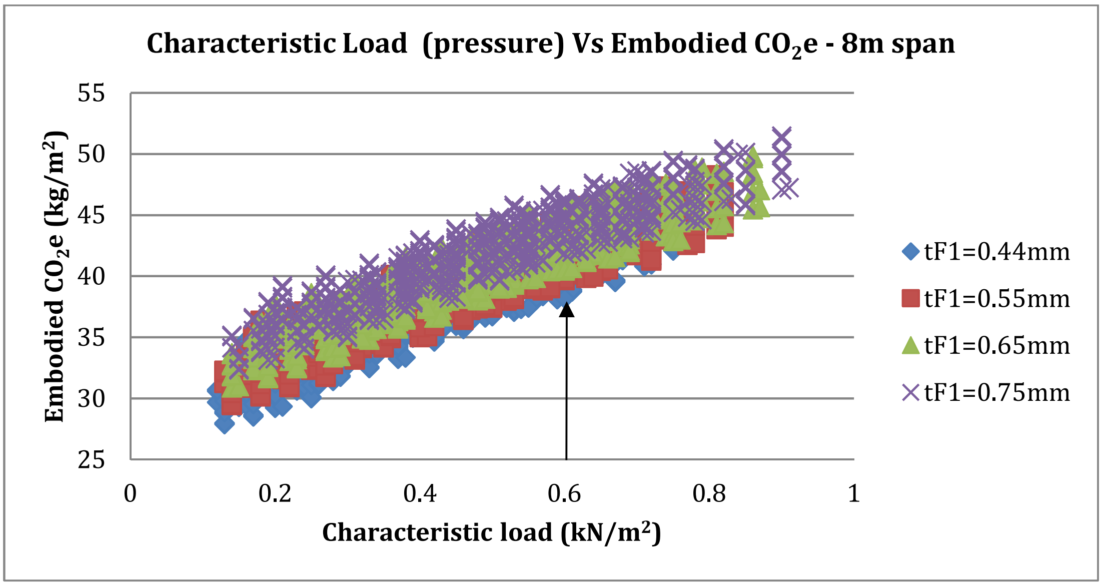


Figure 7.23 Influence of outer sheet thickness on maximum applied load and embodied carbon (pressure)

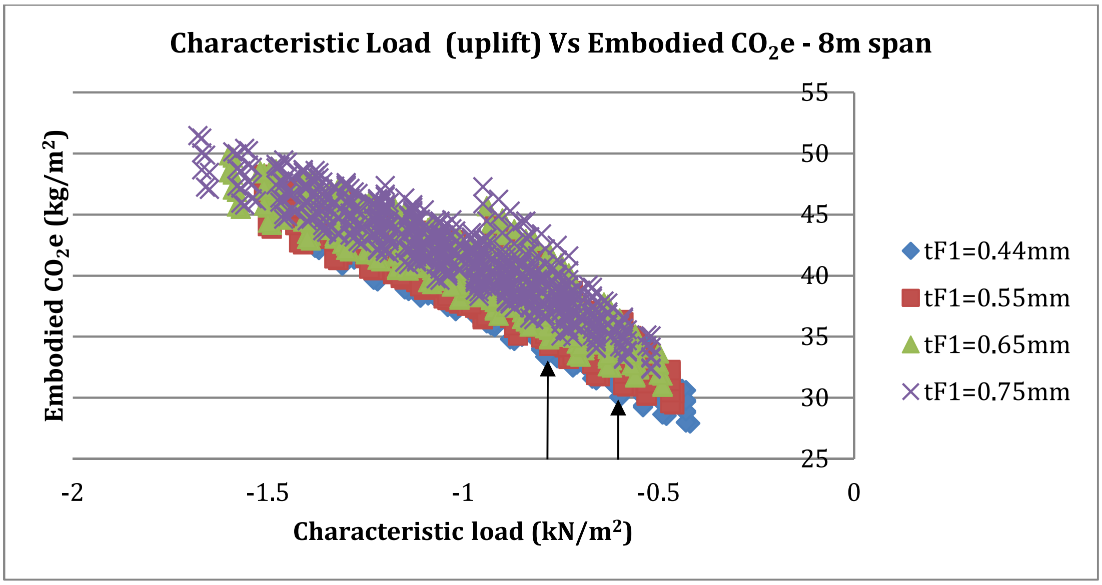


Figure 7.24 Influence of outer sheet thickness on maximum applied load and embodied carbon (suction)

Figure 7.25 and Figure 7.26 show the distribution of solutions based on the thickness of the liner sheet. As for the outer sheet thickness case, as the liner sheet thickness increases, there is a small increase in the allowable characteristic load; however the increase in embodied carbon is minimal. The latter can be explained by the small contribution of the liner sheet within the overall carbon emissions of the panel. The increase in the allowable load is more obvious for the suction case, which is reasonable considering that the increase in the liner gauge leads to improved compressive resistance of the liner sheet, which govern the design in suction load. For pressure case, the improvement due to thicker liner sheet is almost negligible. The Pareto-front for suction case comprises the middle-range of thicknesses, with the exception of the low and high pressure ends of the graph, where the Pareto-front comprises the thinner and thicker values respectively. For the pressure case, the Pareto-front comprises the full range of values. The plots show that for load magnitude required to be resisted in either pressure or uplift, the thinner liner sheet (0.34mm) would be marginally optimal.

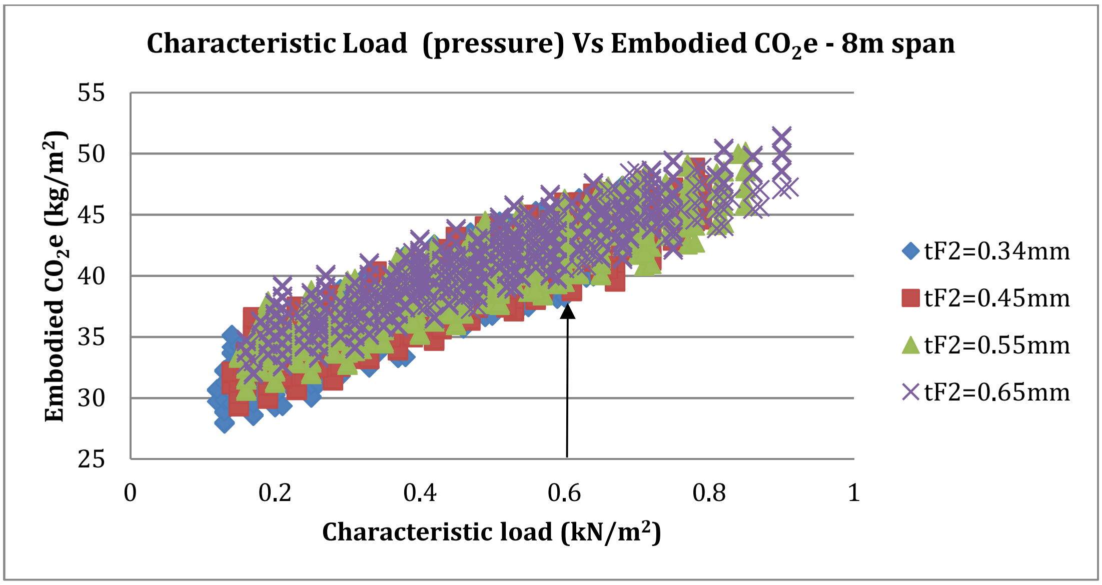


Figure 7.25 Influence of liner sheet thickness on maximum applied load and embodied carbon (pressure)

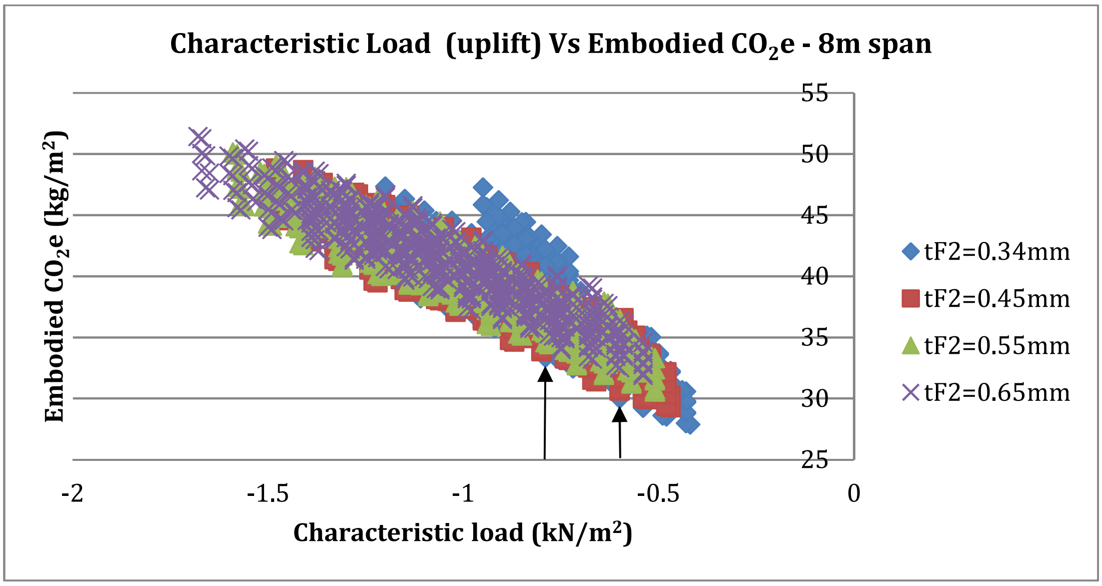


Figure 7.26 Influence of liner sheet thickness on maximum applied load and embodied carbon (suction)

Figure 7.27 and Figure 7.28 show the distribution of solutions based on the width of the liner profile. As discussed in Section 7.4.4, the narrower the profile, the lower the slenderness ratio and, consequently, the higher the compressive resistance. The results clearly show that there is marginal influence in the panel performance in either pressure or uplift. The 23.8mm profile width is marginally optimal for the required maximum allowable loads in pressure and the 0.78kN/m² load in uplift as shown on the plots, while the 33.8mm profile is optimal for the 0.66kN/m² load in uplift.

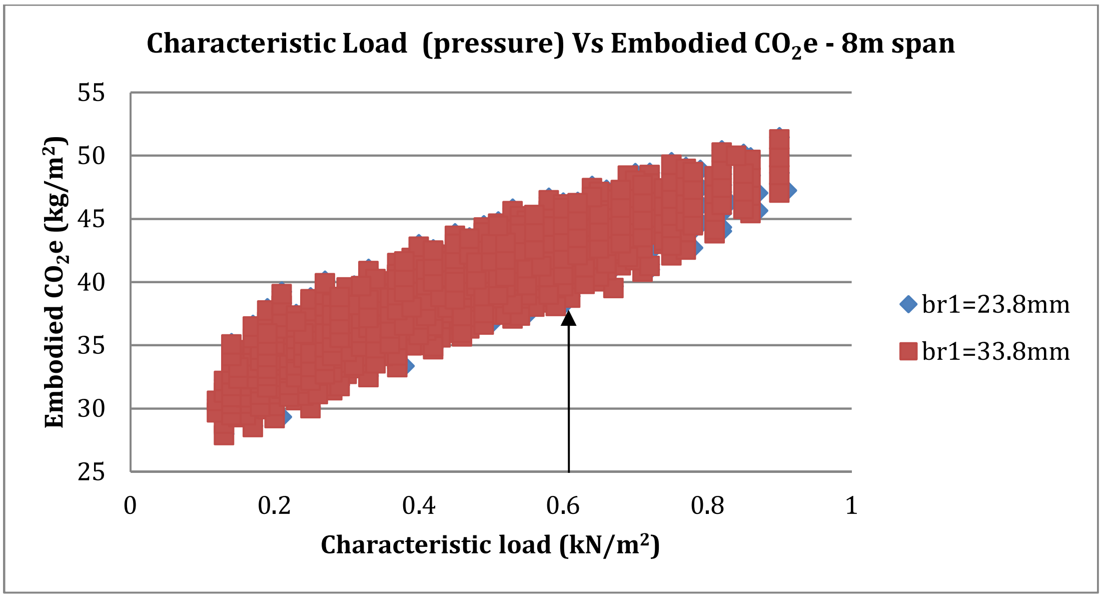


Figure 7.27 Influence of liner width on maximum applied load and embodied carbon (pressure)

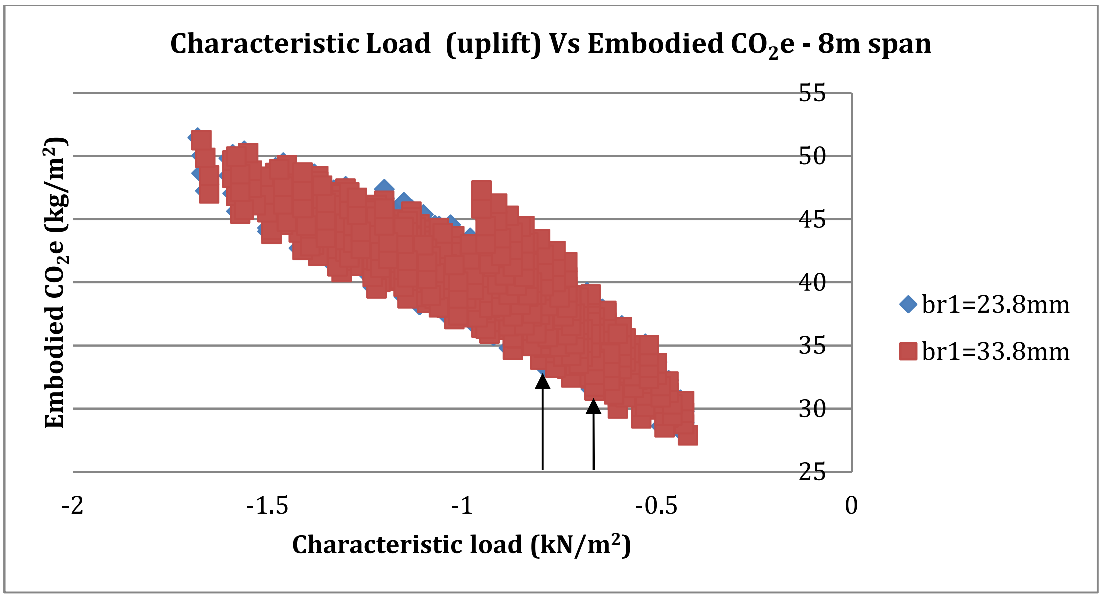


Figure 7.28 Influence of liner width on maximum applied load and embodied carbon (suction)

7.6.9 Concluding remarks and optimal solution

A study was undertaken to revise the specifications of fully profiled sandwich panels with steel faces and PIR cores in order to achieve long-span performance as roof envelope systems for the defined distance and arrangement (8.0m single-span). In Section 7.6.8, the sensitivity and influence of each variable on the development of the optimised solution was examined. These are summarised in Table 7.12, where the optimum variable combinations are shown for each of the required maximum allowable load magnitude in both pressure and uplift.

Table 7.12 Pareto-optimal solutions: variable combinations

Variable	**Imposed load (pressure): 0.6kN/m²**	**Wind load (suction)[1]: -0.66kN/m²**	**Wind load (suction)[2]: -0.78kN/m²**
d_C (mm)	150	100	120
d_R (mm)	61.3	61.3	61.3
b_2 (mm)	23.8	23.8	23.8
ρ_c (kg/m³)	38	38	38
t_{F1} (mm)	0.44	0.44	0.44
t_{F2} (mm)	0.34	0.34	0.34
Embodied CO_2e (kg/m²)	38.213	31.437	33.356
Max pressure (kN/m²)	0.6	0.28	0.38
Max uplift (kN/m²)	-1.11	-0.66	-0.79

[1]*Medium-size building;* [2]*Large-size building*

The results show that the imposed load (pressure) governs the required specifications for the 8.0m long-span sandwich panel. The optimal solution addresses the load requirements in both pressure and wind suction for both medium and large building sizes and, consequently, corresponds to the sought after long-spanning panel solution and associated embodied carbon for the problem in question. The optimal solution shows that the only variables requiring modification against the currently available reference panel are the core depth and the depth of the profile of the outer sheet together with a modest modification of the liner sheet's profile width, while the remaining parameters are the same. This is in agreement with the observations is Section 7.6.8, where the most influential parameters in achieving the optimised solutions for long span capability were found to be the depth of the core and the depth of the outer sheet's full profile. An important observation was that the core density was found to have little effect on achieving the required maximum load; regardless the load magnitude, the lowest density would yield the embodied-carbon optimal solution without compromising the structural performance. Given the very high emissions rate of the PIR, this is an important finding and indicates that future specifications may pursue a reduction in the density of the PIR core.

Table 7.13 summarises the specifications of the optimised long-span sandwich panel derived on the basis of the Pareto-optimal solution and compares them to key specifications and embodied carbon impacts of existing panel products. These include (reference to Table 2.3 and Table 2.9) roof sandwich panels:

- With the backstop U-value to Part L 2010/2013 ($0.25W/m^2K$) – 80mm core depth.

- Complying with the notional building U-value to Part L 2010/2013 (0.18W/m^{2}K) – 120mm core depth (0.16W/m^{2}K)
- With an improved U-value (0.15W/m^{2}K) – 135mm core depth; panel suitable for long span application (6.67m) in small buildings (see Chapter 4).

The optimised long span panel has an improved thermal performance (U-value 0.138W/m^{2}K) when compared to the currently available system with improved U-value (0.15W/m^{2}K).

Table 7.13 Summary of variables for Pareto-optimal solution and comparison to existing panels

Variable	**Currently available panel systems**			**Enhanced panel (Pareto-optimal solution)**
Application	Backstop U-value*	Notional building U-value*	Improved U-value; Long-span (6.67m) for small building	Long span (8.0m) for medium and large building
d_C (mm)	80	120	135	150
d_R (mm)	31.3			61.3
b_2 (mm)	33.8			23.8
ρ_c (kg/m3)	38			38
t_{F1} (mm)	0.44			0.44
t_{F2} (mm)	0.34			0.34
Embodied CO_2 (kg/m^2)**	24.7	31.1	33.6	38.2

**According to Part L 2013/2016; **Using the same methodology as discussed in Section 7.6.2.2*

A comparison in terms of U-values and broken down embodied carbon emissions is shown in Figure 7.29. The figures show that the main increase is due to the PIR material, while the contribution of the carbon emissions due to the additional steel are minimal. The embodied carbon quantification method used was consistent for all panels.

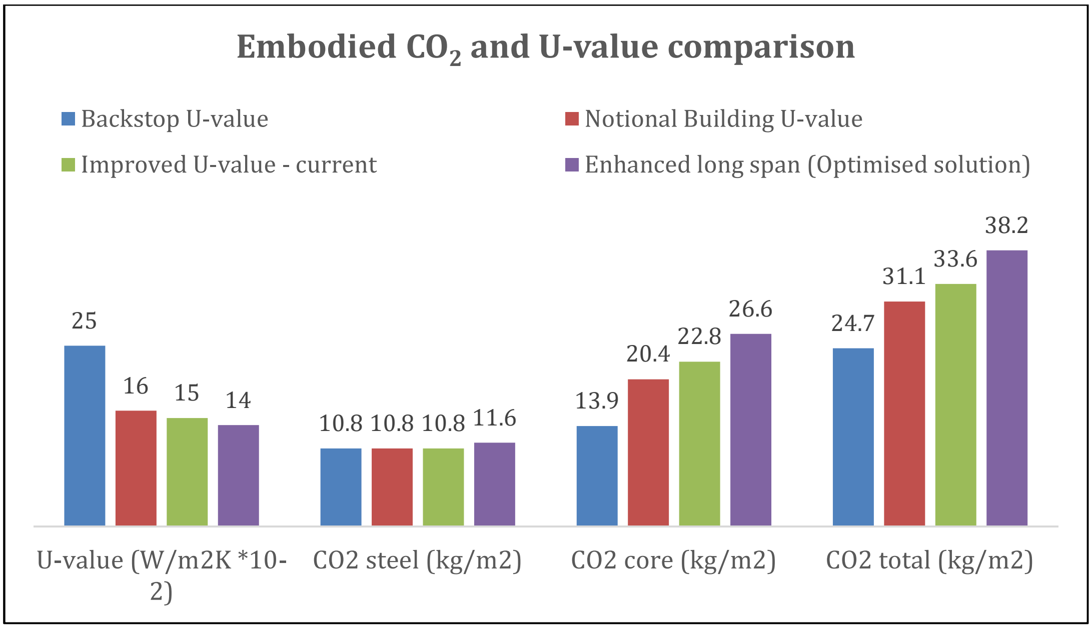


Figure 7.29 Embodied carbon and U-value comparison

The results show that the optimised long span solution has increased embodied carbon by approximately:

- 55% compared to the panel complying with the Part L 2010/2013 backstop U-value
- 23% compared to the panel complying with the Part L 2010/2013 notional building U-value
- 13% compared to the currently available panel with the improved U-value (0.15W/m^{2}K).

The embodied carbon impact derived for the specified long span sandwich panel solution is used in Chapter 8 for the holistic building review.

Chapter 8 Systems review

The feasibility studies presented in Chapter 4, Chapter 5 and Chapter 6 concluded that structural schemes with long span roof systems provide the greatest possibility for reduction of structure in terms of frame material. This is particularly true for trussed-roof steel frames with northlights. The subsequent study in Chapter 7 defined the embodied carbon-optimum long span roof sandwich panel specifications to achieve the optimum span and frame spacing distances derived in Chapter 4.

The current chapter presents a comparative review among the trussed-roof steel frames with northlights combined with the defined long span sandwich panel systems against the typical current practice for frame and cladding specifications. A holistic building review was carried out in terms of embodied carbon and relative cost to include the impact of various systems, comprising frames, envelopes, foundations, rooflights, northlights and flashings. A qualitative review of the impact of structure on the operational carbon emissions of the buildings is also presented.

8.1 Structural schemes

The structural schemes for the systems' review were selected so that a comparison could be made between the current practice for single storey industrial buildings (i.e. portal frames with purlins and sandwich panels or built-up roof systems) against the option of trussed-roof steel frame with northlights and long span roof sandwich panels, defined in Chapter 4 as the most promising opportunity to yield material reduction.

The following structural options were assessed:

- Duo-pitch portal frames with purlins at optimum spacing and with sandwich panel roofs (complying with defined thermal requirements)
- Duo-pitch portal frames with purlins at optimum spacing and with built-up roofs (complying with defined thermal requirements)
- Trussed-roof steel frames with northlights at optimum spacing and enhanced long span sandwich panels where required
- Trussed-roof steel frames with northlights with maximum roof sandwich panel spans achievable by current technology (complying with defined thermal requirements).

The optimum frame spacing for each building size and structural scheme were according to Table 4.9 in Chapter 4. A summary of the structure-envelope options and their reference system are shown in Table 8.1.

Table 8.1 Structural schemes, roof envelope systems and frame spacing options for systems review

Structural scheme	Roof envelope system	Frame spacing	Option reference
Duo-pitch portal frames with purlins	Sandwich panels – conventional ($U=0.15W/m^2K$)	Optimum for each building size	1A (Base Case)
	Built-up ($U=0.15W/m^2K$)	Optimum for each building size	1B
Trussed-roof steel frames with northlights	Sandwich panels – enhanced (optimised) ($U=0.14W/m^2K$)	Optimum for each building size	2A
	Sandwich panels – conventional ($U=0.15W/m^2K$)	Maximum allowable with current technology	2B

It is highlighted that the enhanced roof sandwich panel system defined in Chapter 7 has a U-value of $0.14W/m^2K$, which is slightly better than the U-value of $U=0.15W/m^2K$ of the current available technology (reference panel in Appendix A). The U-value of $0.15W/m^2K$ was defined through the literature search as that anticipated for future energy conservation requirements.

The frame spacing distances for each building size are shown in Table 8.2. It should be noted that for the case of small buildings, the optimum frame spacing for the trussed roof frames and the maximum allowed by the current sandwich panel technology coincide.

Table 8.2 Frame spacing distances for systems review

Frame spacing	Small building (1-bay)	Medium building (2-bay)	Large building (2-bay)
Optimum	6.67m	8.00m	8.33m
Maximum allowable with current technology	6.67m	6.67m	6.57m

8.2 Embodied carbon appraisal

8.2.1 Modelling

As the operational carbon efficiency of buildings increases, so does the relative importance of the construction materials' and processes' embodied carbon. A study of the embodied carbon was, therefore, undertaken to assess the environmental implications of the selected structural schemes and envelope systems and in particular to compare the options for long span opportunities with those for traditional portal frame construction.

Embodied carbon assessments are very sensitive to and highly dependent on the assumptions made and the source data. As Target Zero (2011) recommends, transparency of the input is vital for an appropriate interpretation of results.

The full assessment of the embodied carbon of a development requires the identification of the lifecycle greenhouse gas emissions (expressed as carbon dioxide equivalent) occurring during the following stages:

- Manufacture and transport of construction materials
- Construction process
- End-of-life stages (demolition, disposal, recycling, re-use)

The system boundaries selected for the study comprise a 'cradle-to-gate' approach, i.e. transportation of materials to site, construction processes and end-of-life options are excluded. This was judged to be sufficient for the comparative nature of the present study for the following reasons:

- The stages included in the 'cradle-to-gate' approach yield a high level of certainty and remain unchangeable for the manufactured product regardless of its treatment once it leaves the manufacturing gate. On the other hand, transportation, construction and end-of-life stages vary significantly, depending on the project. Furthermore, any subsequent stages can be processed separately and their impact can be combined with the 'cradle-to-gate' quantities for further study where required.
- Transportation of materials to and from the fabrication, construction and waste sites is highly project-specific and depends on the site's location which may vary significantly. Furthermore, there is currently no common database for the transport effects of the whole range of materials and building elements required for the present study. Overall, it was judged that robust assumptions for the transport effects could not be made within the generic context of the appraisal.
- There is no robust database for energy and carbon required during construction process, hence it is very difficult to quantify unless a full construction economics study was undertaken. This would demand a significant effort to create robust data, would demand very generic and questionable assumptions and it would

overall be outside the scope of the study. Furthermore, according to Target Zero (2011a, 2011b) for cases of warehouse and supermarket buildings, construction processes accounted for less than 1% of the total embodied carbon. Hence it was judged that this stage can be excluded from the present comparative study with confidence.

- Moreover, it is acknowledged that fabrication of different structural systems such as portal frames and trusses would require different levels of energy. However, anecdotal information and consultation with members of the steel industry suggest that the energy associated with fabrication is significantly lower than the energy associated with materials and steel component manufacture. Hence, this stage was not included in the study either.
- There is no standard industry approach for the end-of-life options and there is lack of a common database for the end-of-life impacts of the various materials used on the study (steel, concrete, plastics, glass, insulation). The steel construction sector advocates the 'cradle-to-grave' approach, while the concrete sector argues against it. There are many unresolved issues considering the end-of-life options which still require significant research.
- The selected embodied carbon database explained below comprised a 'cradle-to-gate' approach while it guaranteed consistency within the study and a single reference source for the wide range of materials used. Overall, it was selected that end-of-life options are excluded from the present study.

The embodied carbon of the selected options was appraised using the Inventory of Carbon and Energy (ICE) database from the University of Bath (Jones and Hammond, 2008). The database is well-established and freely available, while major organisations, such as the Environment Agency, incorporate it within their carbon calculation tools. Furthermore, the ICE incorporates a single database and transparent methodology for the whole range of construction materials included in the present study. Finally, the same database was utilised in Chapter 7 for the embodied carbon analysis for sandwich panels as per manufacturer's practice. Hence, by using the ICE for the whole building assessment, consistency among the construction materials and components is ensured. The embodied carbon coefficients for the materials used in the study are shown in Appendix F.

As discussed in Section 7.6.2.2, it is important to highlight that the assumed energy used in manufacture and the consequent embodied carbon emissions of materials and products could change substantially in the future as the energy grid efficiency and source mix change. Hence, it is likely that the embodied carbon of materials included in Bath's ICE database is subject to major changes in the future. As the energy grid emissions in the UK and rest of Europe are reducing and fossil fuel participation reduces in the energy source mix, it is likely that material and product emissions are reduced in the future.

In order to enable a fair comparison between the different schemes, only the components with varying specifications and quantities among the options were included in the analysis. The following building elements were included in the embodied carbon appraisal:

- Structural frame (primary structure, purlins, tie-rods)
- Roof
- Walls (external)
- Rooflights
- Northlights
- Flashings
- Foundations

Due to the comparative nature of the appraisal, an assessment of a complete development was outside the scope of the present study. Connection components and fittings (plates, bolts, cleats and fasteners) were excluded from the analysis since their contribution in terms of material weight and, consequently, embodied carbon is generally very small. Furthermore, certain elements were excluded from the analysis because their quantities and specifications are normally project-specific, difficult to generically define and they would not vary between the schemes. Such items included external works, drainage, over-cladding (such as rain screens or dado walls), internal walls, ceilings, finishes, internal fit-out, lifts, doors, windows and building services.

It should be highlighted that external works may have a significant contribution to the embodied carbon of a complete development. Target Zero (2011a, 2011b) showed 18%-21% for a specific warehouse and supermarket development. Similarly, the drainage system was shown to have a 4%-6% contribution.

The ground floor slab and ground fill materials for the sub-base would also be identical for each building size, hence excluded from the initial comparative appraisal. However, their impact is presented towards the end of the review in order to illustrate their relative importance within the building context.

A summary of the specifications for the various components is given in Table 8.3.

Table 8.3 Specification summary for components in systems review

Component type	Specification summary	Reference
Frame (primary and secondary members)	Specified in Chapter 4.	Appendix C
Enhanced roof sandwich panel	Specified in Chapter 7. U=0.14W/m^{2}K, 150mm PIR, 0.44/0.34 S350 steel faces, fully profiled (61.3mm)	Chapter 7
Current roof sandwich panel	U=0.15W/m^{2}K, 135mm PIR, 0.5/0.4 S220 steel faces, fully profiled (31.3mm)	Appendix A – Section A.1
Wall sandwich panel	U=0.20W/m^{2}K, 120mm PIR, 0.7/0.5 S220 steel faces, micro-ribbed	Appendix A – Section A.2
Roof built-up system	U=0.15W/m^{2}K, 300mm Mineral wool, R32/LP1000 0.7/0.4 S220 steel faces	Trisobuilt System (TATA Steel, 2014)
Rooflights	15% of roof area (base case) 20% of roof area (for sensitivity study) Polycarbonate, 3.6kg/m^2, U=1.3W/m^{2}K for sandwich panel roof Glass Reinforced Polymer (GRP), triple-skin, 3kg/m^2 outer, 1.83kg/m2 inner, 40mm polycarbonate insulation, U=1.4W/m^{2}K for built-up roof	Trilite Ultra - EnergySaver (Brett Martin, 2009) GRP Rooflight Range (Hambleside Danelaw, 2013)
Northlights	Covering the full truss depth (16%-32% depending on Option and building size) Multiwall polycarbonate, Clear 3.4kg/m^2, U=1.6W/m^{2}K, Light Transmittance 64% (base case) Double glass, Clear, 4mm toughened outer, 6.4mm laminated low E inner, U=1.6W/m^{2}K, Light Transmittance 73%-75% (for sensitivity study)	Fivewall 25mm, Clear S – Marlon LongLife (Brett Martin, 2010) Ritchlight (Brett Martin, 2010)
Flashings	0.7mm, girth: 250mm at drip, 375mm at ridge, 500mm at gable	AECOM (2015)
Foundations	C40 concrete	
Ground floor slab	200mm C40 concrete, 40kg/m^2 reinforcement, 225mm aggregate sub-base	Knapton (2003)

The specifications of the primary and secondary structural steel components were obtained from the results of the long span study (Chapter 4) and the reference to the section sizes can be made in Appendix C. The specification of the enhanced sandwich panel roof system was obtained from the optimisation study in Chapter 7. The specification of the current sandwich panel system is shown in Appendix A. A typical lightly profiled sandwich panel wall cladding system was spanning directly between columns was selected as common among all the options. The specification of the system is shown in Appendix A. Flashings of 0.7mm steel gauge and L-shaped geometries were used for the analysis according to manufacturer's guidance.

The specification of the rooflights and northlights varied across the different schemes. A polycarbonate system was used for the composite panel roof and triple-layer Glass Reinforced Plastic (GRP) with an insulating polycarbonate layer was assumed for the built-up roof, as in standard practice. Rooflights were assumed to cover 15% of the roof, as in the Part L 2013 (HM Government, 2014) recommendations for the notional building. The scenario of 20% roof covering was also examined as part of a sensitivity study. For the northlights case, the polycarbonate system was used as the base case, while a double-glass system was examined as part of a sensitivity study. The northlights area was dependent upon the truss depth specified (assumed to cover the full truss depth) and it varied between 16.4% and 32% for various building sizes and options. All the glazing options were specified based on yielding a minimum U-value of 1.4W/m^{2}K to comply with the Part L 2013.

Foundations were designed according to BS EN 1997 assuming a typical ground bearing stress of 250N/mm^2. Specifications of the ground floor slab and the sub-base were obtained from Knapton (2003), where the design of a typical industrial floor slab is shown.

8.2.2 Results and discussion

The results of the comparative embodied carbon analysis are shown in the present section.

8.2.2.1 Total embodied carbon

The total embodied carbon impacts for the various options are shown in Figure 8.1, Figure 8.2 and Figure 8.3. Figure 8.4, Figure 8.5 and Figure 8.6 show the normalised results in relation to the floor area of each building.

Relative to the base case for traditional portal frame construction with sandwich panel roof cladding (Option 1A) and for each building size (small, medium and large respectively):

- Option 1B showed higher embodied carbon (+6.3%, +7.3%, +7.1%)
- Option 2A showed lower embodied carbon (-1.8%, -5.0%, -5.3%)
- Option 2B showed varying embodied carbon differences (-1.8%, -0.9%, +4.0%)

Relative to the traditional portal frame construction with built-up roof cladding (Option 1B) and for each building size (small, medium and large respectively):

- Option 1A showed lower embodied carbon (-6.1%, -6.8%, -6.5%)

- Option 2A showed lower embodied carbon (-7.2%, -11.4%, -11.5%)
- Option 2B showed lower embodied carbon (-7.2%, -7.6%, -2.8%)

Option 2A shows the highest savings in terms of embodied carbon in all building sizes. Also, the normalised embodied carbon impact shows an increase as the building size increases.

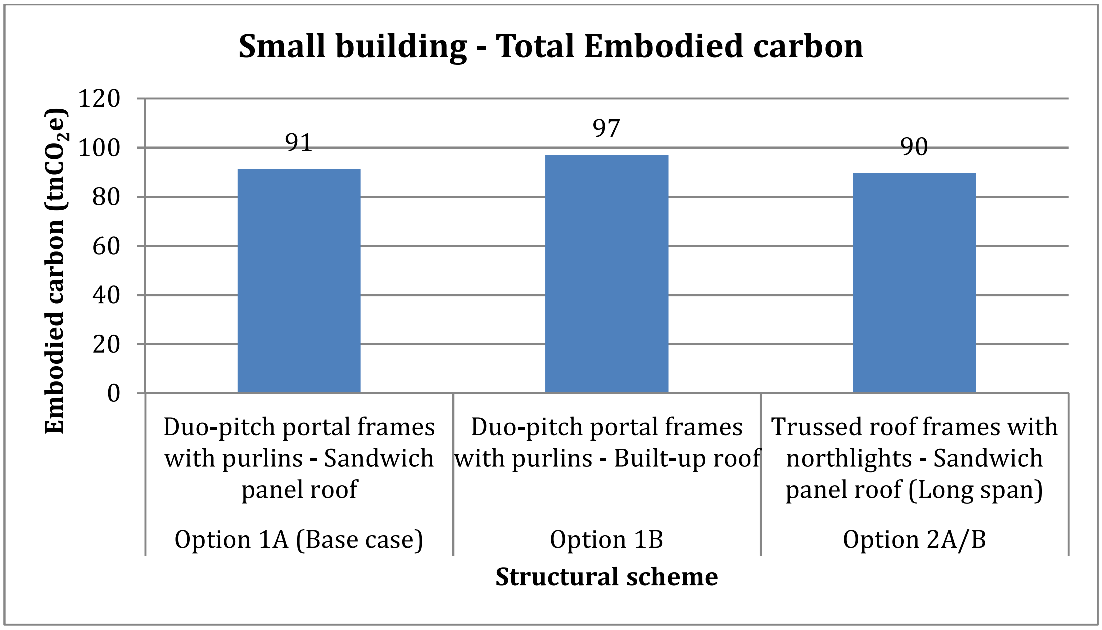


Figure 8.1 Total embodied carbon – Small building

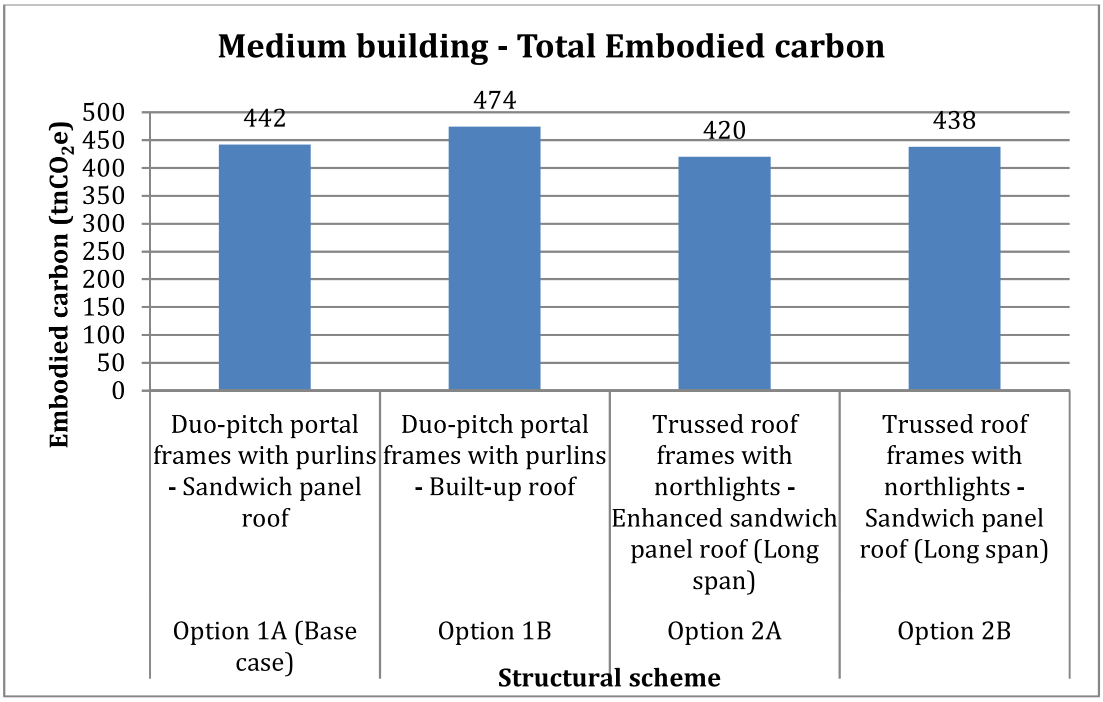


Figure 8.2 Total embodied carbon – Medium building

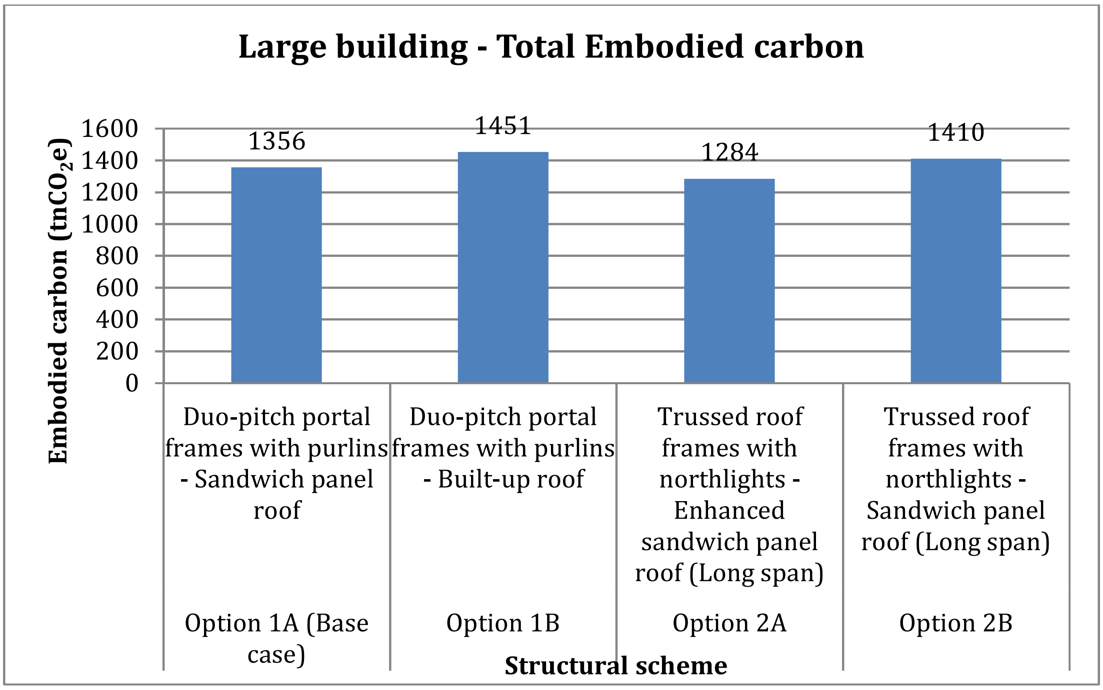


Figure 8.3 Total embodied carbon – Large building

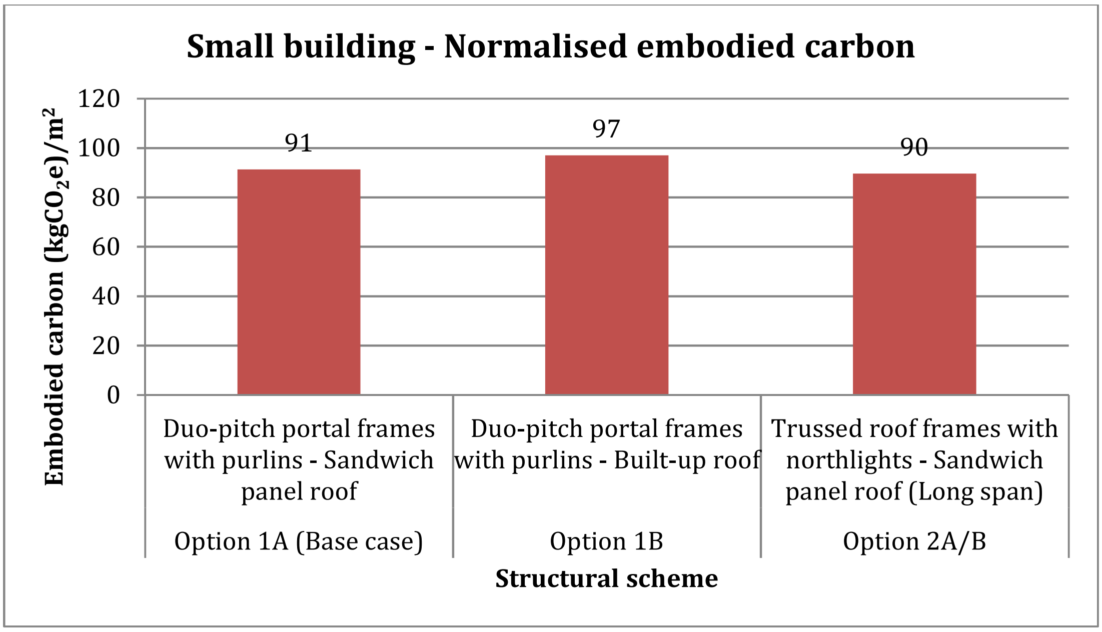


Figure 8.4 Normalised embodied carbon – Small building

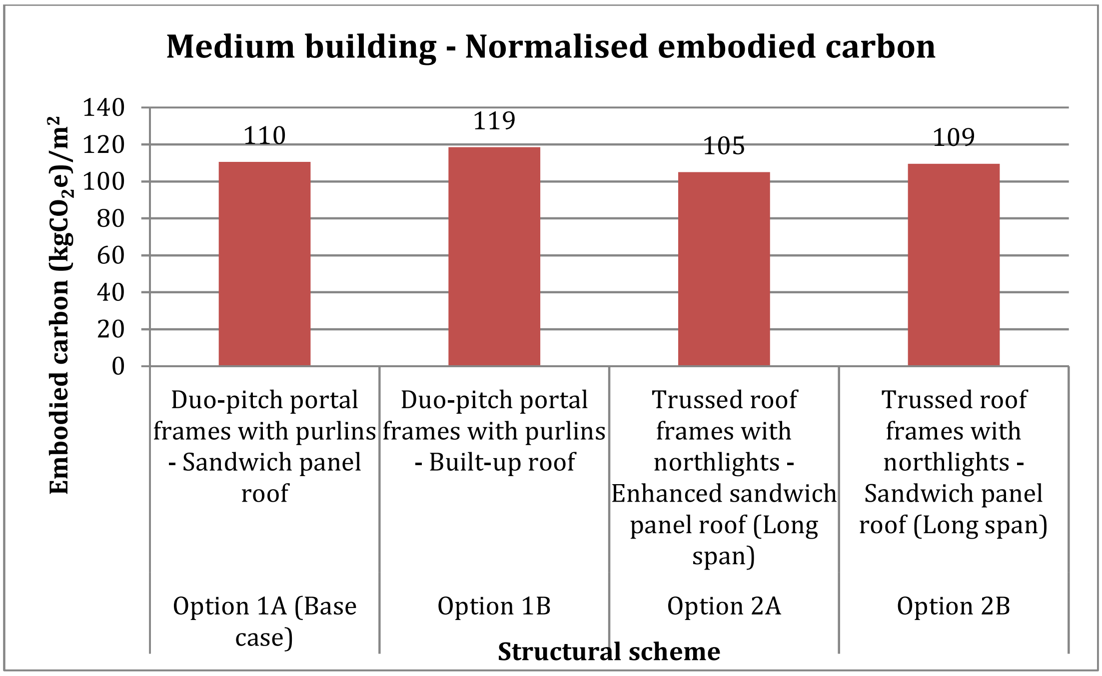


Figure 8.5 Normalised embodied carbon – Medium building

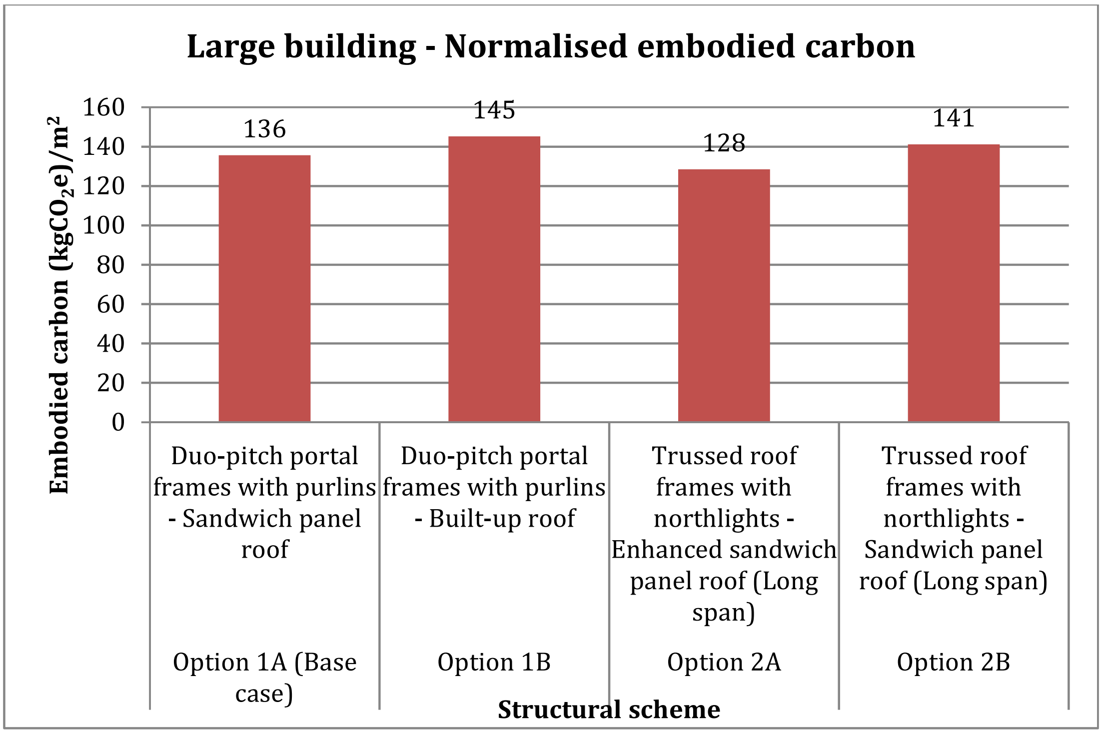


Figure 8.6 Normalised embodied carbon – Large building

8.2.2.2 Mass breakdown

The mass of materials used for each option are broken down by element in Figure 8.7, Figure 8.8 and Figure 8.9 and by material in Figure 8.10, Figure 8.11 and Figure 8.12. The following key observations were made:

- Most of the materials are used in the foundations (67%-94% depending on building size and scheme) which overwhelm by far the frame (2%-15%) and cladding (1%-8%). The percentage of the foundations' contribution also increases with the size of the buildings, while frames and cladding percentages decrease.
- Concrete is also used in 67%-94%, followed by steel in 5%-25% and much smaller mass of insulation (PIR or Mineral wool), 1%-8%. As for the elements' breakdown case, the contribution of concrete within the total mass increases with the building size, unlike steel and insulation which decrease. The glazing elements and materials showed very small contribution within the total building mass.
- The heaviest option for all building sizes was Option 1B. This is because the weight of the portal frames is higher than the trussed-roof frames and also because the built-up roof system is heavier than the sandwich panel. The trussed-roof options require larger roof cladding; however the total weight is still smaller than the built-up roof option.
- The results also indicate that the lighter trussed-roof frames require heavier roof cladding due to the larger roof area of Options 2A and 2B and the heavier specifications of the enhanced panel for Option 2A.
- The heavier frame-envelope assemblies also had a negative impact on foundations.

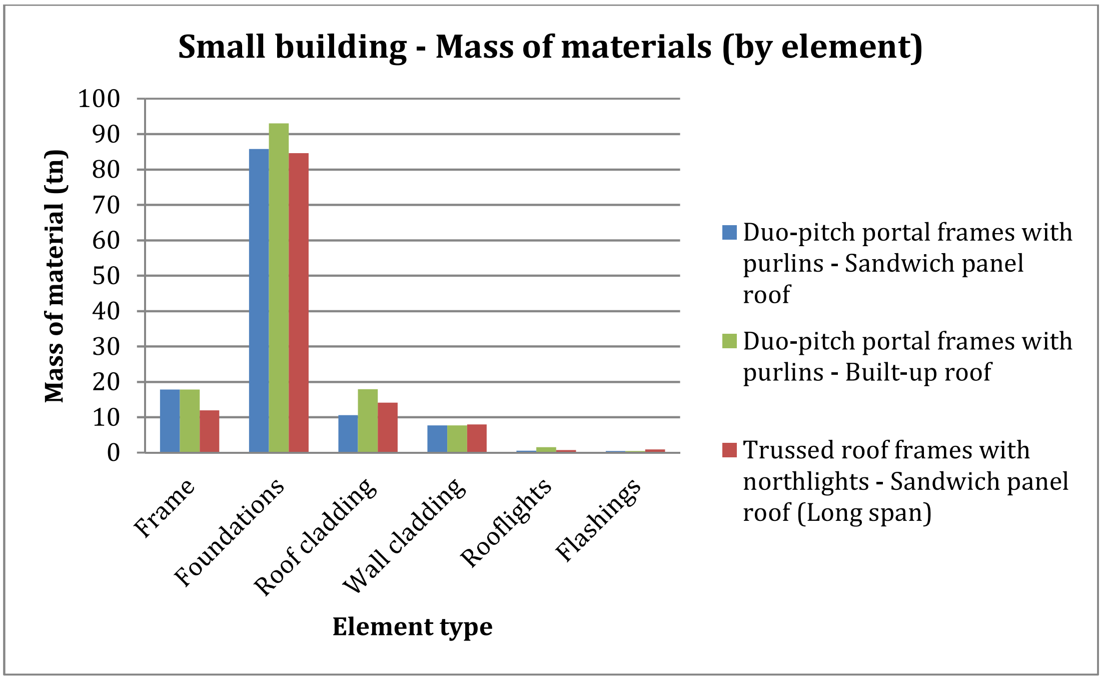


Figure 8.7 Breakdown of mass of materials by element – Small building

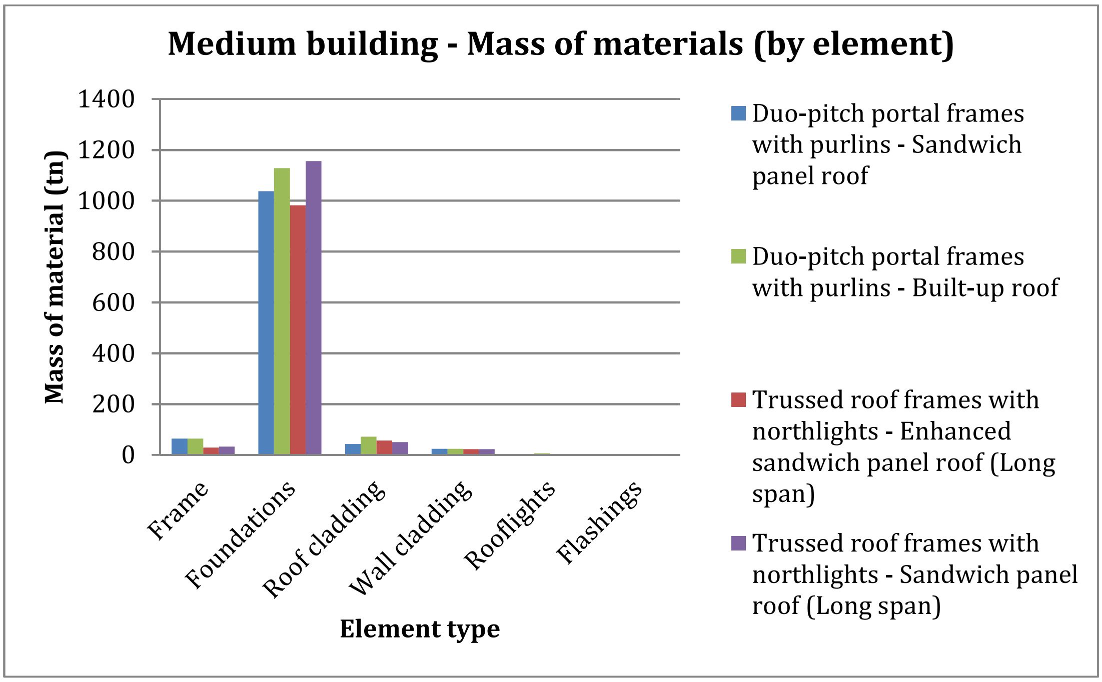


Figure 8.8 Breakdown of mass of materials by element – Medium building

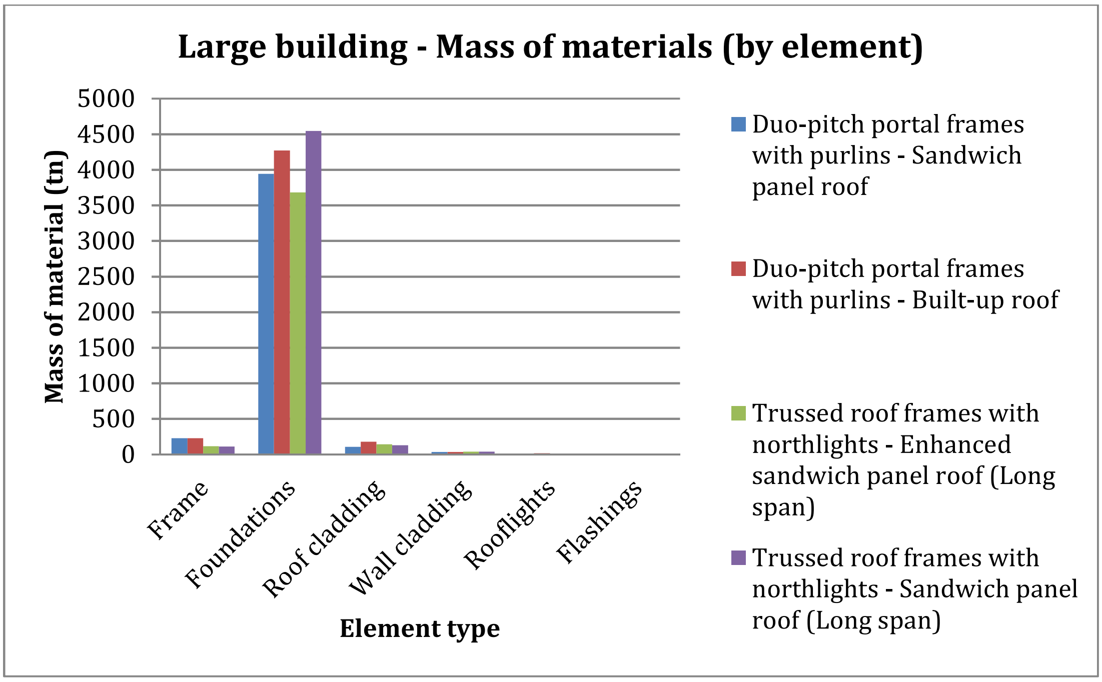


Figure 8.9 Breakdown of mass of materials by element – Large building

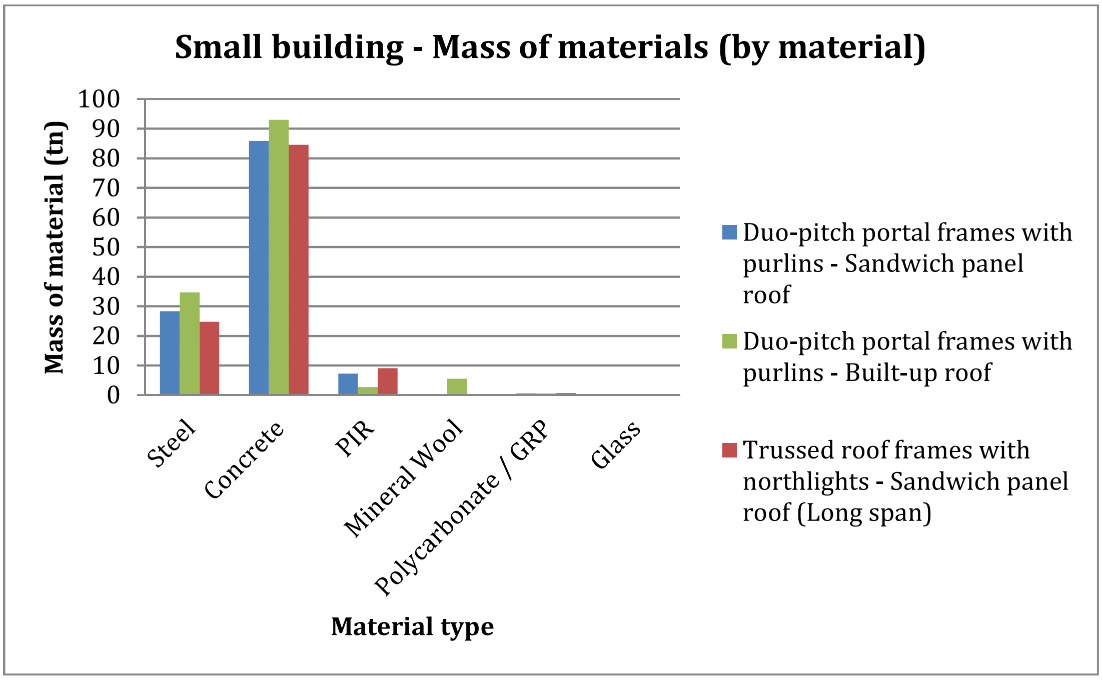


Figure 8.10 Breakdown of mass of materials by material – Small building

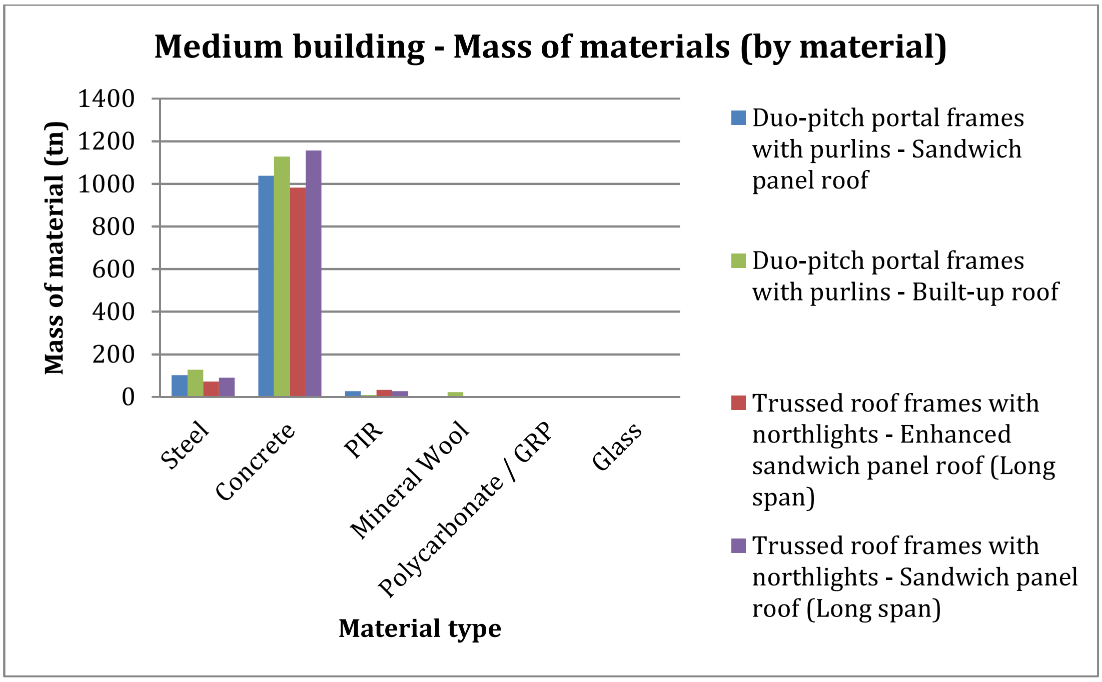


Figure 8.11 Breakdown of mass of materials by material – Medium building

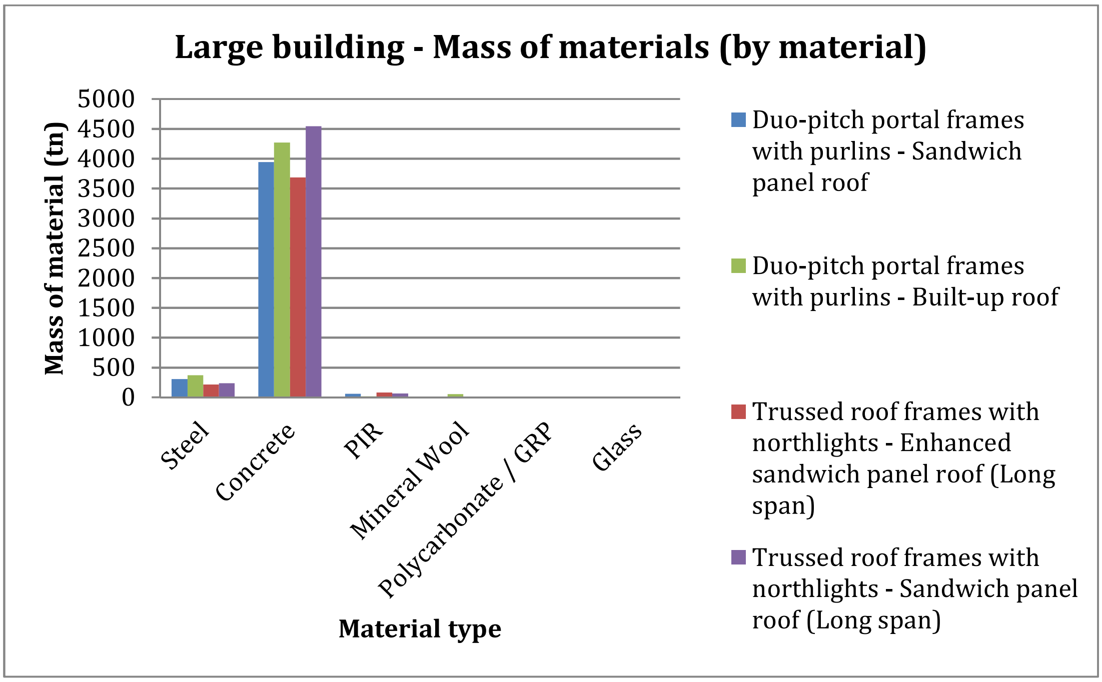


Figure 8.12 Breakdown of mass of materials by material – Large building

8.2.2.3 Embodied carbon breakdown

The breakdown of embodied carbon for the various options and building sizes is given in Figure 8.13, Figure 8.14 and Figure 8.15 by element and Figure 8.16, Figure 8.17 and Figure 8.18 by mass.

From Figure 8.13, Figure 8.14 and Figure 8.15 showing breakdown by element, the following key observations were made:

- The largest contribution came from the roof cladding for the small building (27%-38%), while for the medium and large came from the foundations (35%-49%).
- Option 2 showed significantly less embodied carbon for the frame, due to the lower frame weight compared to Option 1. Concurrently, there was higher roof cladding impact due to the larger roof area and also due to the heavier enhanced sandwich panel for Option 2A. Option 2A with optimum frame spacing was found to have less embodied carbon for the frame and roof combined compared to Option 2B which made use of the current sandwich panel envelope technology.
- There is an observable impact of the heavier superstructure on the foundations. Also, the increased number of frames in Option 2B demanded more footings, a consequence which is illustrated as increased foundation weight in the medium and large buildings.
- The wall cladding impacts were similar across all options; however a small increase was shown for Option 2 due to the marginal increase of wall cladding area required for the formation of northlights slopes.
- There was also a considerable contribution of the embodied carbon of the rooflights and northlights. The GRP option for built-up roofs was found to have a considerable impact, unlike the polycarbonate options for the sandwich panel roofs and northlighs which had a much smaller impact. On the same time, the increased northlights area caused higher glazing embodied carbon burden for Option 2A and 2B, with the latter having higher impact.
- Flashings were found to have a small impact overall, which makes the variation among the schemes trivial.

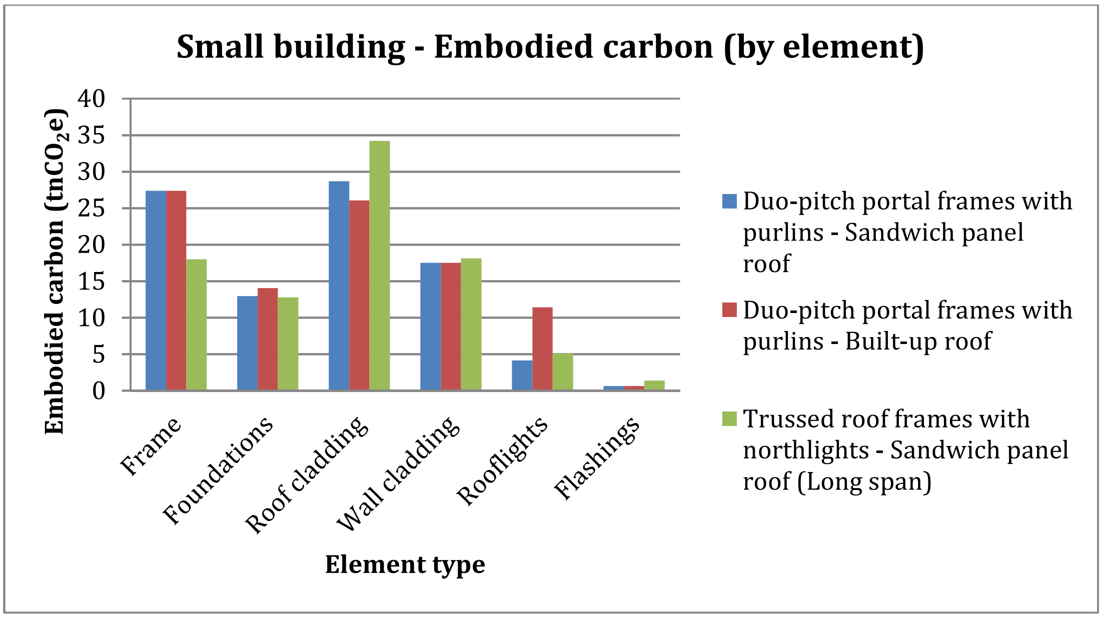


Figure 8.13 Breakdown of embodied carbon by element – Small building

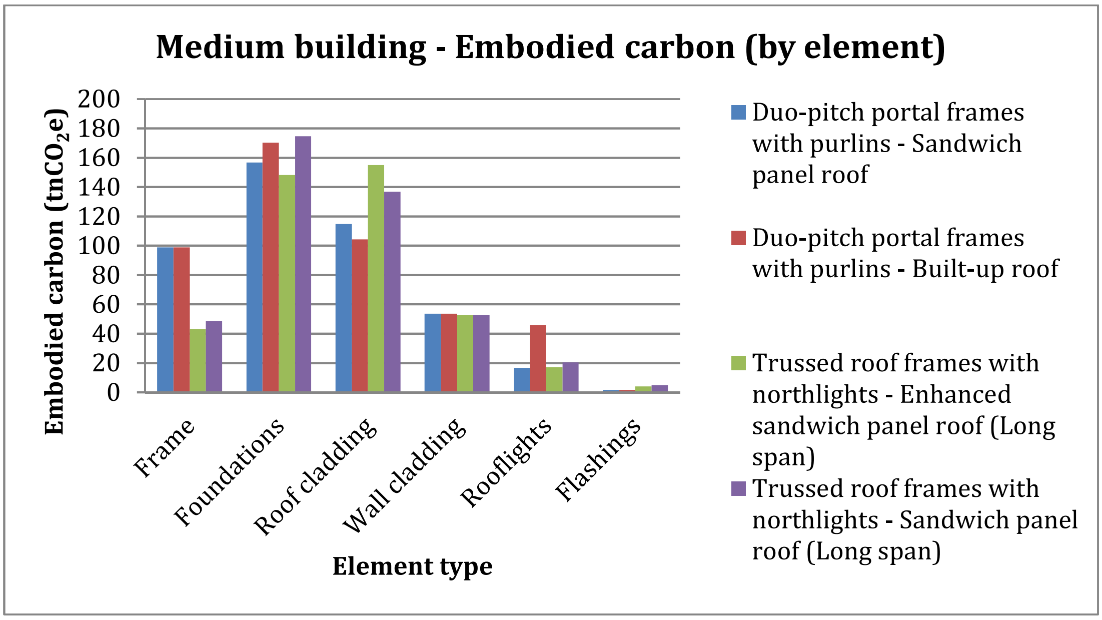


Figure 8.14 Breakdown of embodied carbon by element – Medium building

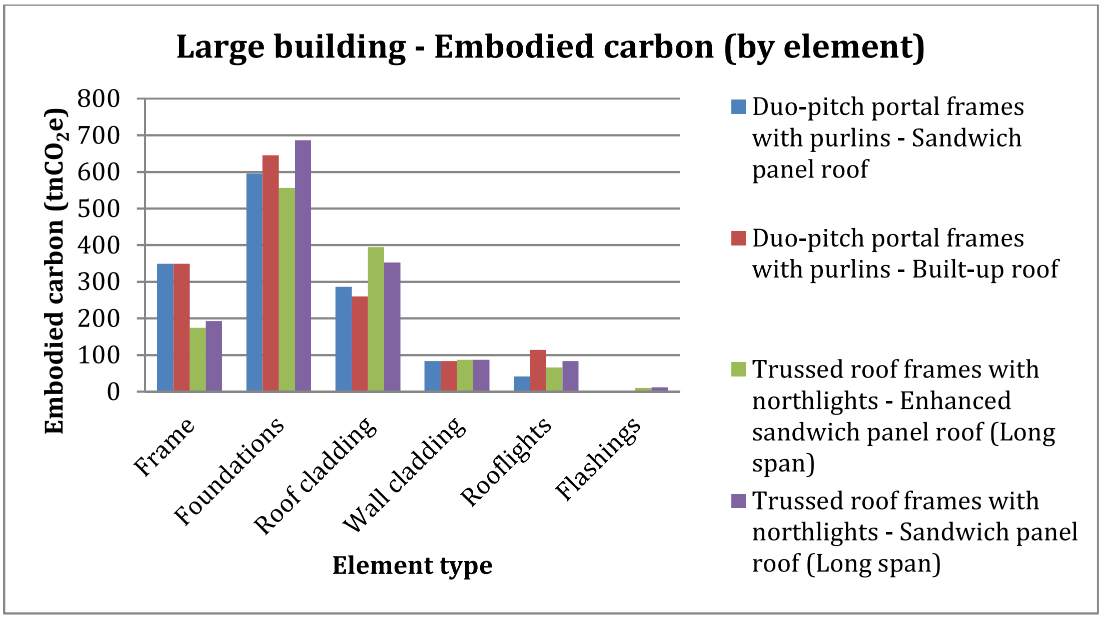


Figure 8.15 Breakdown of embodied carbon by element – Large building

From Figure 8.16, Figure 8.17 and Figure 8.18 showing breakdown by material, the following key observations were made:

- Concrete is the greatest contributor of embodied carbon in the large building, while steel is in the small building. In the medium building, the material with the greatest contribution is either steel or concrete and depends on the scheme. It is important to note that the embodied carbon associated with the steel is not only due to the structural frame but also the steel sheeting used for the roof and wall coverings. The built-up roof, for example, has higher impact due to steel because of the thicker steel sheets used.
- PIR has a significant impact due to its very high emissions per tonne.
- In terms of insulation materials, it is apparent that the mineral wool options have a much lower impact compared to the PIR options, due to their low embodied carbon per tonne and despite the increased mass compared to PIR.
- The impact of rooflights and northlights is small but should not be disregarded. GRP options for built-up roof assemblies showed a higher impact compared to the polycarbonate options for rooflights and northlights.

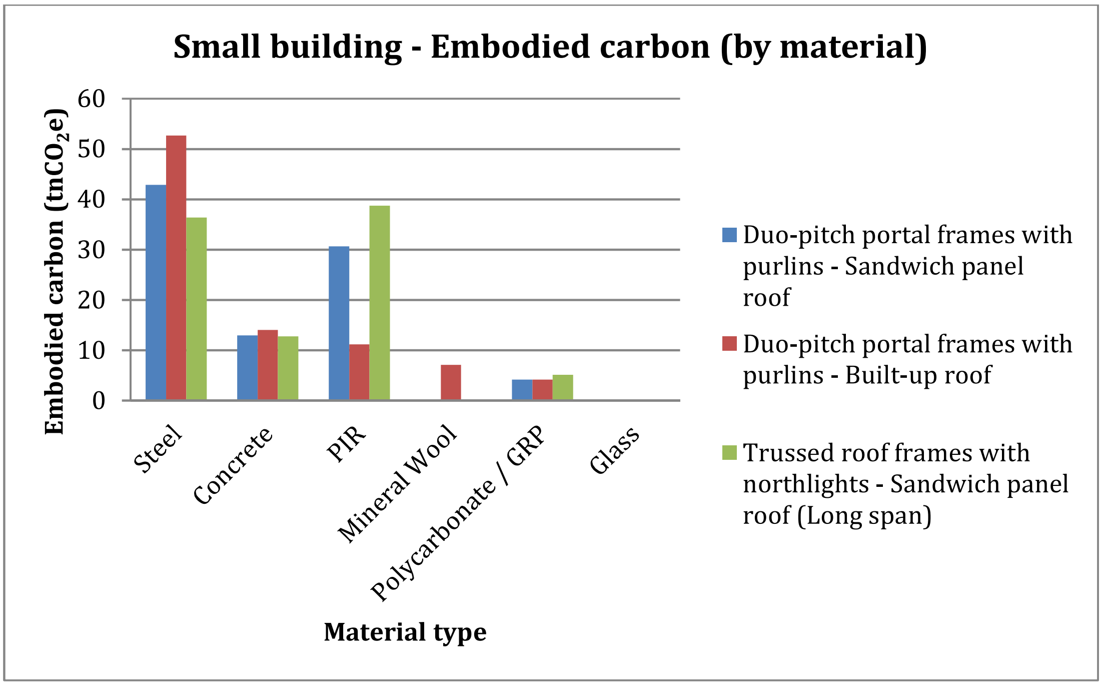


Figure 8.16 Breakdown of embodied carbon by material – Small building

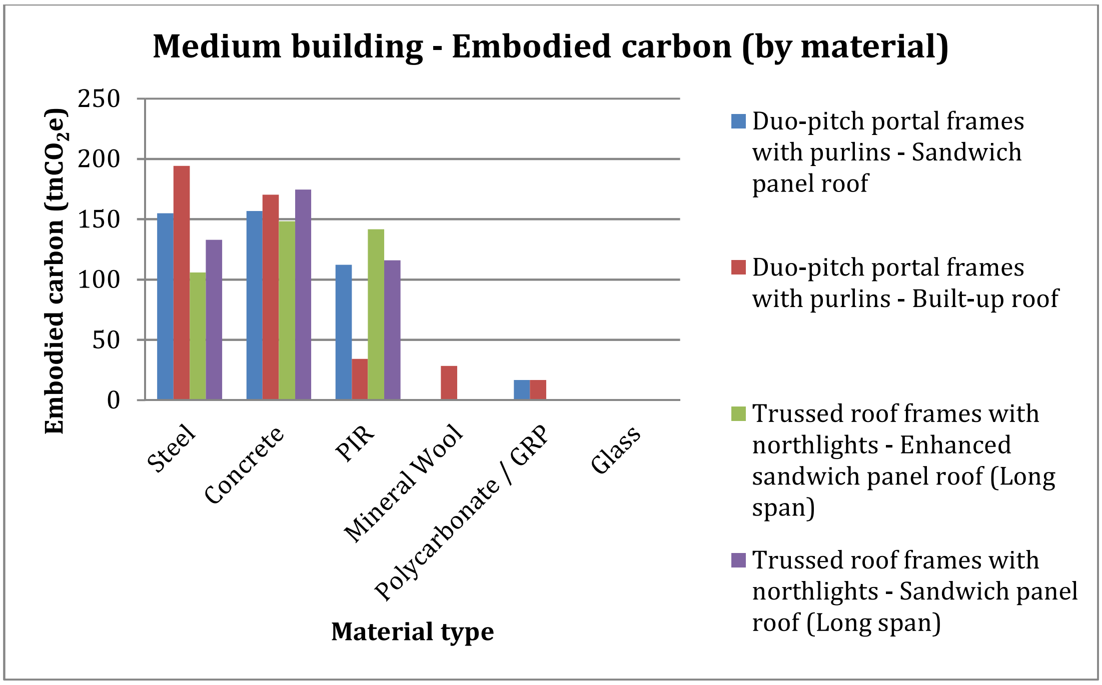


Figure 8.17 Breakdown of embodied carbon by material – Medium building

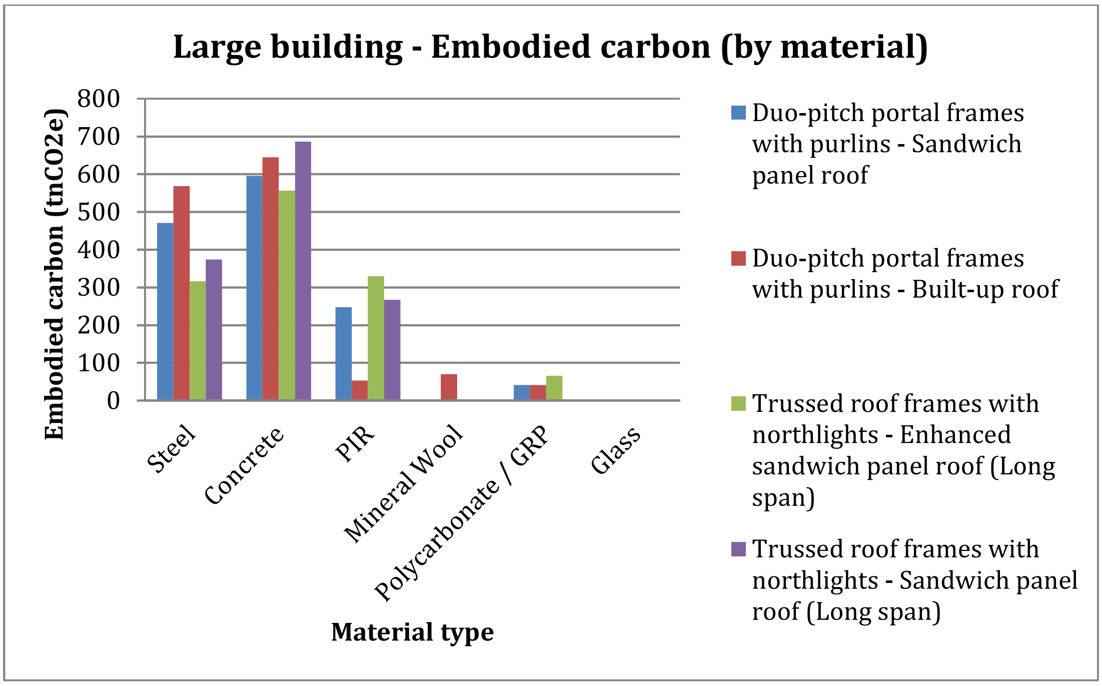


Figure 8.18 Breakdown of embodied carbon by material – Large building

8.2.2.4 Impact of floor slab and sub-base

The floor slab and the sub-base were not included in the initial analysis since they do not vary across the schemes for each building size. However, it was found that their contribution to the overall building embodied carbon was significant. Figure 8.19 shows the breakdown of the embodied carbon of the various materials of the ground floor slab and sub-base, as well as their combined impact per building size. Compared to the total embodied carbon of the super-structure and foundations shown in Figure 8.1, Figure 8.2 and Figure 8.3, it is easy to notice that the impact of the ground floor slab is almost of the same amount as when all the other components are added together. This is due to the high volume of concrete, which despite its very low emissions rate per unit mass, yields an overwhelming total embodied carbon impact. The impact of steel reinforcement is very small within the total impact of the slab, while the aggregate filling is minimal due to the extremely low embodied carbon rate and despite its high volume.

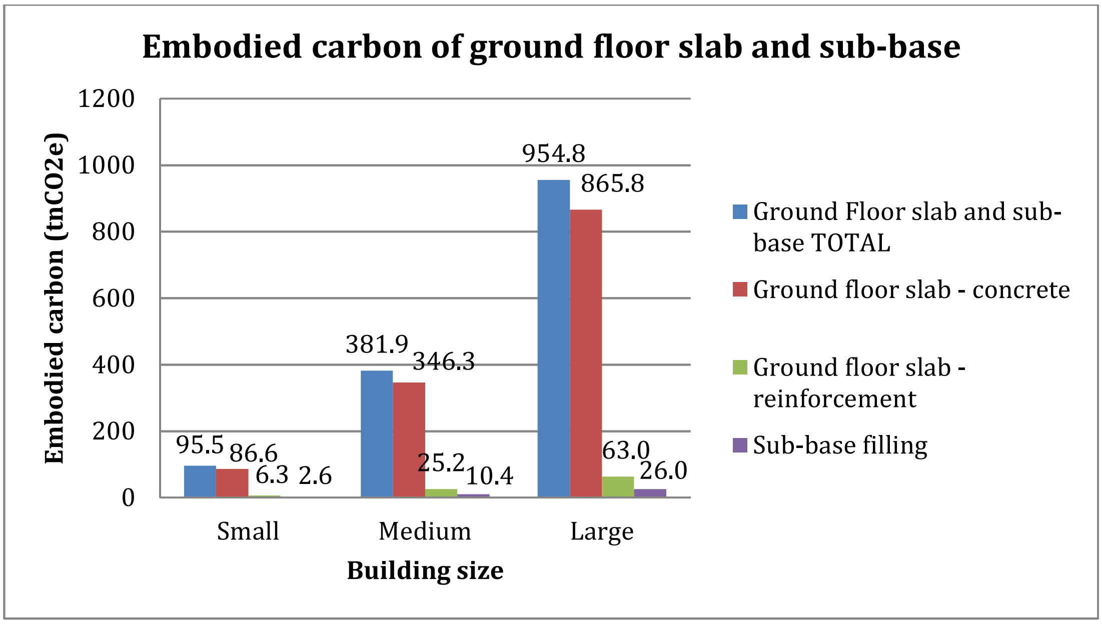


Figure 8.19 Embodied carbon of ground floor slab and sub-base (total and breakdown)

8.2.2.5 Impact of rooflights area

Figure 8.20, Figure 8.21 and Figure 8.22 show the variation of the total embodied carbon when the percentage area of rooflights changes from 15% to 20%. When the figures are compared to Figure 8.1, Figure 8.2 and Figure 8.3 it may be noticed that the embodied carbon of Option 1A decreases (<0.2%) while for Option 1B increases (<2%). This is because the embodied carbon of the polycarbonate system used for sandwich panel roofs is lower compared to the sandwich panel roof system per unit area, hence a part of the roof with higher impact is substituted with components of lower impact. For the built-up roof case the situation is different. GRP rooflight systems have a higher impact compared to the roof cladding, hence a proportion of lower embodied carbon roof is substituted by higher impact components. This observation is useful since the rooflights area may vary depending on the operational energy performance and optimisation requirements.

There is obviously no impact on the trussed-roof frames since the area of northlights is dictated by the truss height. Also, the trussed-roof options show still lower total embodied carbon.

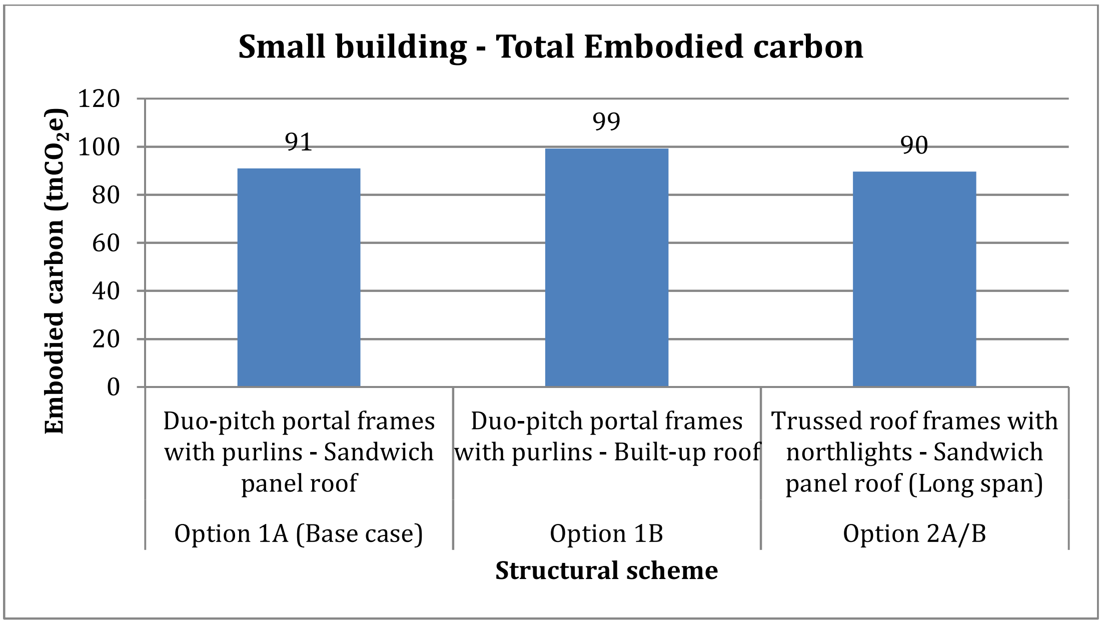


Figure 8.20 Total embodied carbon - Small building with 20% rooflights

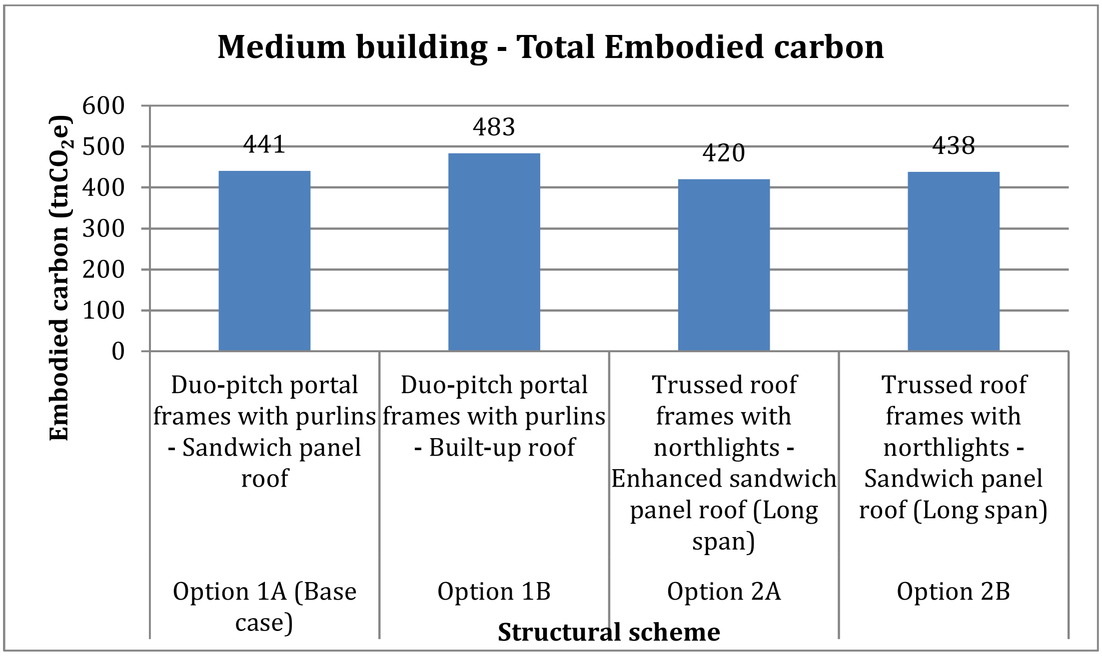


Figure 8.21 Total embodied carbon - Medium building with 20% rooflights

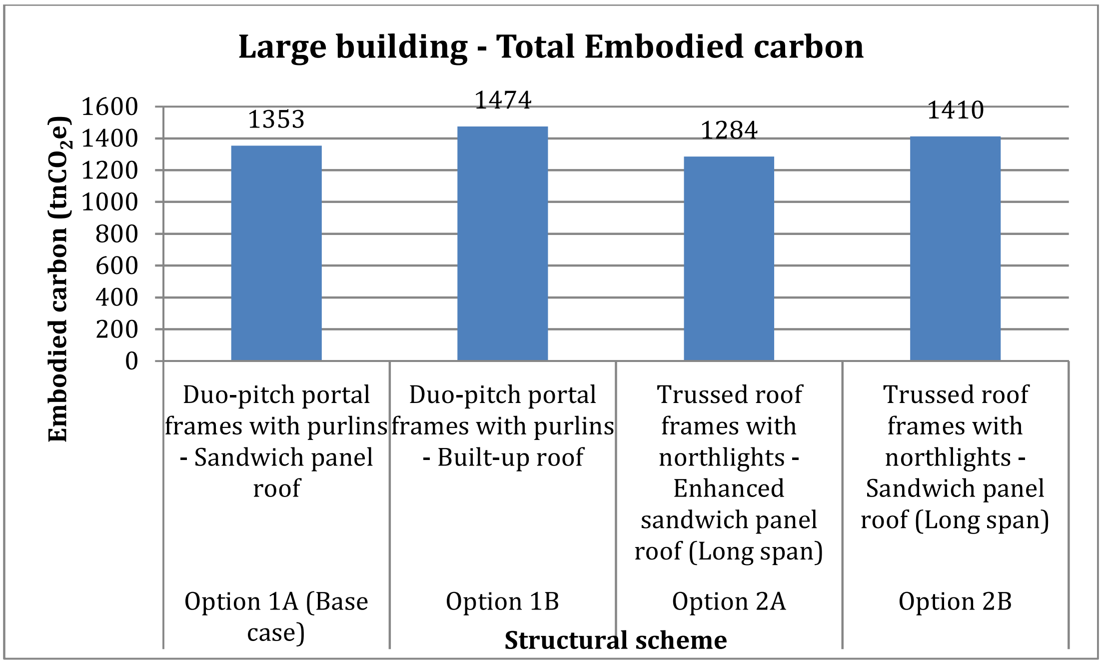


Figure 8.22 Total embodied carbon - Large building with 20% rooflights

8.2.2.6 Impact of northlights glazing type

Figure 8.23, Figure 8.24 and Figure 8.25 show the total embodied carbon when the glass option instead of the polycarbonate is used for the northlights. A comparison with Figure 8.1, Figure 8.2 and Figure 8.3 shows that an increase of the total embodied carbon occurs for the glass option (1.1%-2.1%). This is because the glass system has a much higher weight. Although the impact of glass per unit mass is much lower than the polycarbonate, the extra weight creates a negative embodied carbon balanced. Furthermore, the increased weight has a negative effect on the foundations, which are required to be designed for higher load and, consequently, higher volume is needed. Nevertheless, the increase in terms of total embodied carbon is very small overall.

Despite the change, Option 2A remains the scheme with the lowest total embodied carbon, with the exception for the small building case, where the same impact with Option 1A is shown. This is still the case when the figures are compared with the 20% rooflight option in Figure 8.20, Figure 8.21 and Figure 8.22.

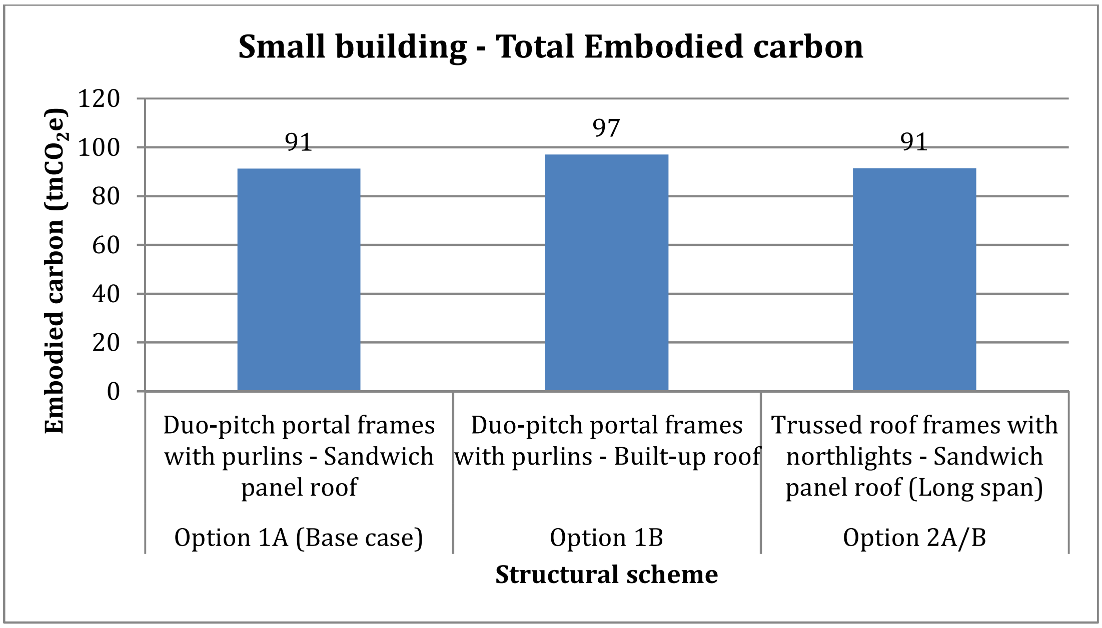


Figure 8.23 Total embodied carbon - Small building glass northlights

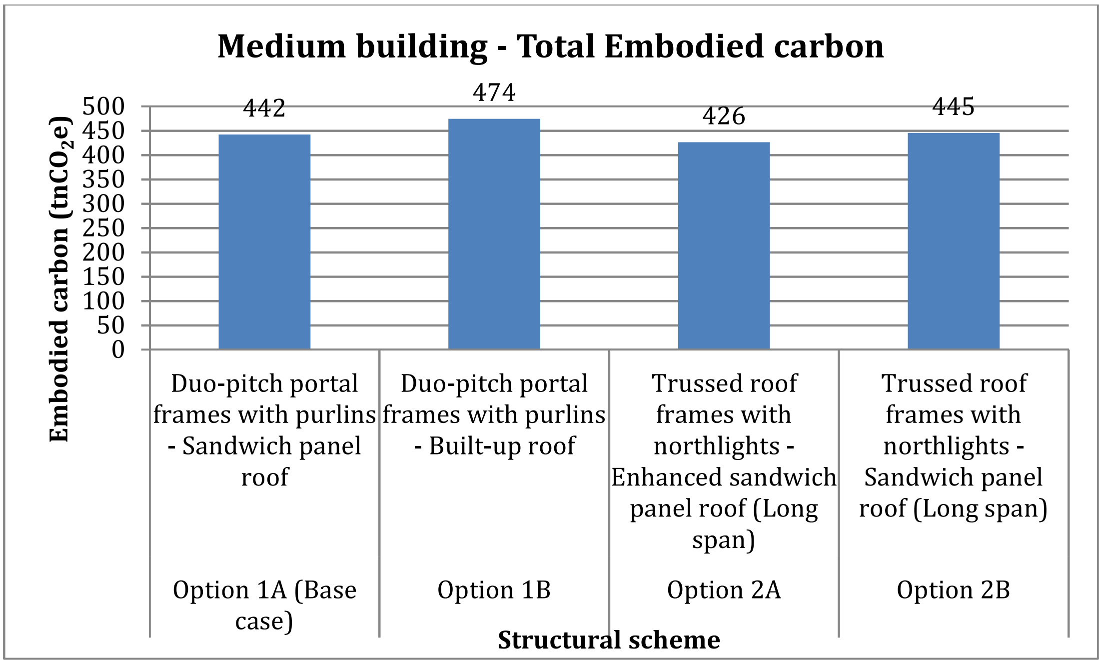


Figure 8.24 Total embodied carbon - Medium building with glass northlights

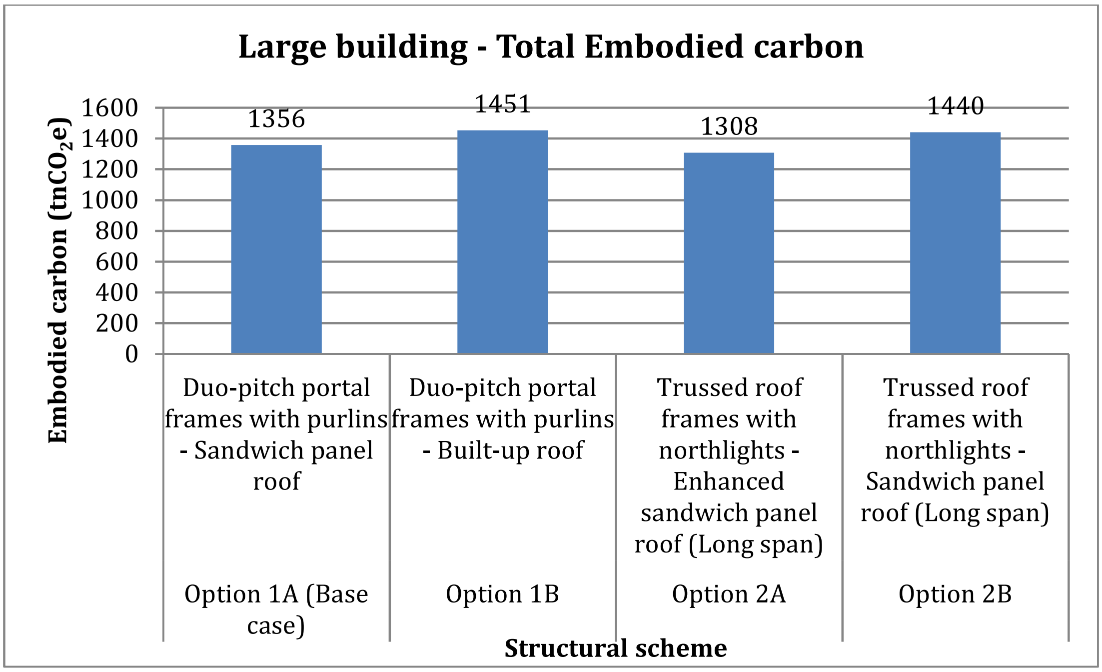


Figure 8.25 Total embodied carbon - Large building with glass northlights

8.3 Impact of structure on operational carbon emissions

Option 1 and Option 2 are fundamentally different in terms of design and operational energy performance. The differences concern the following parameters which affect various energy conservation and operational energy aspects (in brackets):

- Internal volume of building (energy for heating and cooling)
- Rooflight / northlight system arrangement and position (light distribution)
- Rooflights / northlights area (heat losses, light distribution)
- Thermal transmittance (U-value) of the glazing material / component (heat losses)
- Light transmission of the glazing material / component (light intensity)
- Length of interfaces (heat losses, air-tightness).

In order for the performance of the building in terms of operational energy to be fully assessed, the use characteristics of each building would firstly need to be defined and a dynamic thermal modelling would be required. The present study considers single storey buildings from a structural point of view without looking at a specific uses, e.g. warehouses, supermarkets, cold-storage etc. which have different operational energy requirements. It is outside the scope of the present study to analyse the operational energy and carbon performance of the defined buildings and various scenarios of uses. However, appreciation was given to the fact that the different structure-envelope assembly options will have different impacts on the operational carbon performance. Hence, a qualitative appraisal was undertaken, based on results and recommendations discussed in Section 2.2.4.3.

As discussed in Section 2.2.4.3, the use of northlights is ideal to reduce cooling, solar gains and avoid overheating in the building, while it is has lighting benefits (uniformity, consistency and intensity for large glazing areas) compared to rooflights. Where cooling is not required, rooflights are probably better in terms of beneficial solar gains; however, there is a high risk of overheating.

In the present study, the truss depth for Option 2 was chosen such that the building volumes for Options 1 and 2 remain the same. Consequently, a larger glazing area (16%-32% of the roof) was specified for Option 2 compared to the rooflights area (15% of the roof) in Option 1 (see Table 8.4). The same volume between the options indicates that heating requirements would be the same volume-wise. However, as Option 2 has a larger glazing area, the heat losses will be higher, hence more energy for heating will be required. Heat losses will also be higher due to the fact that the roof area in Option 2 is slightly larger than in Option 1. The other generic aforementioned benefits of northlights in terms of cooling, ventilation and avoidance of overheating are also anticipated.

Table 8.4 Glazing areas (% of roof)

Building size	Option 1A	Option 1B	Option 2A	Option 2B
Small	15%		19.7%	
Medium			16.4%	16.7%
Large			25.2%	32.0%

The increased glazing area in Option 2 would also allow more light into the building, reducing the energy required for lighting compared to Option 1. As discussed earlier, lighting is expected to dominate vastly the operational energy demand, especially for buildings which do not require cooling.

The truss span to depth ratio was 19.1 for the small and medium buildings and 15.3 for the large building. Typical practical limits for truss span-to-depth ratios are 15:1 for light loads to 10:1 for heavier loads, although these ratios highly depend on the relative importance of steel material and fabrication costs. If the northlights glazing height and area were smaller, then a reduced truss depth would be required. That would vary the lighting gains and the thermal losses compared to the current scheme. Furthermore, it would require re-design of the structure to account for the geometry modification, leading to different amount of materials, hence embodied carbon impact.

The current frame spacing for Option 2 also gives better roof slopes (see Table 8.5) and ideal orientation for potential installation of PVs, compared to the small slopes for Option 1, although these are not close to the optimal recommendation of 30°-35°. Decreasing the frame spacing would increase the roof slope for Option 2 and if PVs were about to be installed, their output would be closer to the optimal as the roof slope increased. On the other hand, closer frame spacing would be expected to yield higher steelwork weights, hence higher embodied carbon. Moreover, longer interfaces requiring joint detailing and flashings would be required for the increased amount of frames; therefore, more sources of thermal and air leakage would be present.

Table 8.5 Roof slopes

<table>
<tr><th>Building size</th><th>Option 1A</th><th>Option 1B</th><th>Option 2A</th><th>Option 2B</th></tr>
<tr><td>Small</td><td colspan="2" rowspan="3">6°</td><td colspan="2">11.1°</td></tr>
<tr><td>Medium</td><td>9.3°</td><td>11.1°</td></tr>
<tr><td>Large</td><td>15.5°</td><td>17.5°</td></tr>
</table>

Finally, in terms of the northlights glazing systems selected for the study, the polycarbonate system has a lower light transmission (68%) than the glass one (73%-75%). Hence, a lower lighting performance with the polycarbonate system would be anticipated. However, as it was earlier shown, the glass option led to higher embodied carbon impact for both the glazing and the structure in total. It should also be highlighted that the polycarbonate and glass systems generally achieve much better (lower) U-values compared to GRP rooflights which are used for the built-up roof case in Option 1B.

Overall, it is impossible to assess the exact impact of the structural options on the operational carbon emissions of the building without the use of dynamic thermal modelling and assumptions of the buildings' use parameters. However, there are some compelling arguments based on previous literature, that the northlights options may have significant benefits particularly in terms of cooling and lighting energy savings, as well as enhancing the output of potential PVs on the roofs. A parametric study considering the various structural parameters (truss/glazing height, frame spacing, material type) as well as climates and their impact on both the operational and embodied carbon emissions to define net carbon savings or losses would be a good scope for future studies.

8.4 Comparative construction cost appraisal

8.4.1 Modelling

A comparative construction cost appraisal was carried out to assess the cost variation among the different options. Given its comparative nature, the study was undertaken based on weights of materials and including the following aspects:

- Primary steelwork materials
- Primary steelwork fabrication
- Frame (primary and secondary steelwork) erection
- Secondary steelwork material and fabrication, including fit-outs (purlins, tie-rods, cleat and bolts)
- Foundations materials and placement
- Building envelope (roofs, walls, rooflights, northlights, flashings) materials and fabrication

A full construction cost study of a full development was not within the scope of the current appraisal, in a similar manner as for the embodied carbon study. Construction programme and transportation would normally be project-specific and outside the scope of the comparative study. Hence, these aspects were excluded from the analysis.

The specifications of the components used in the analysis were identical to those in the embodied carbon appraisal and summarised in Table 8.3.

The rates were based on the up-to-date values in Spon's Architects' and Builders' Price Book 2015 (AECOM, 2015). For the secondary steelwork and the building envelope components in particular, where identical products to those assumed for the study could not be found in the handbook, reasonable assumptions were made and the rates for the closest products were chosen. Hence, it is important that the results of the cost appraisal are treated as generic and indicative, acknowledging that they depend upon the stated assumptions. The cost rates for the components used in the study are shown in E.2.

It is acknowledged that effects of off-site construction would have an important impact on construction cost. Such effects included transportation economy and reduced part counts to be installed on site, since installation costs are related to the number of components. Off-site effects are partially included in the study in relation to the reduced steelwork weights occurring for the long span schemes. However, a full construction economics study would be required to assess the full effects, which is out of the present scope. Hence, it was decided to indicate off-site construction benefits in terms of component parts count among the various schemes.

It should be highlighted that transportation of long span truss systems and rafters was taken into account by considering the number of segments. Transport of components less than 18.3m does not require police notification and escorting and is generally the preferable option to deliver pre-fabricated components on site. Excess of that length requires police notification and escorting if up to 27.4m and advanced notification to the Ministry of Transport in excess of 27.4m (BCSA, 2003). For the purpose of the study, it was assumed that trusses would need to be broken into segments of less than 18.3m for the small and medium buildings and assembled on site, while rafter would not exceed that limit either. For the larger buildings it was assumed that truss segments and rafters could exceed 18.3m, requiring a police notification and escorting from the fabricator to the construction site.

8.4.2 Results and discussion

8.4.2.1 Total cost

Figure 8.26, Figure 8.27 and Figure 8.28 show the normalised costs per unit floor area for each option and building size. Relative to the base case (Option 1A) and for each building size (small, medium, large respectively):

- Option 1B showed increased cost by +0.5%, +0.7%, +0.8%
- Option 2A showed lower cost by -10.0%, -23.9%, -26.8%
- Option 2B showed lower cost by -10.0%, -26.6%, -26.2%

Overall, Option 2 was found to be the most economical in all cases. Option 2B was the most economical for the medium building and Option 2A for the large building, with a marginal difference between the two options.

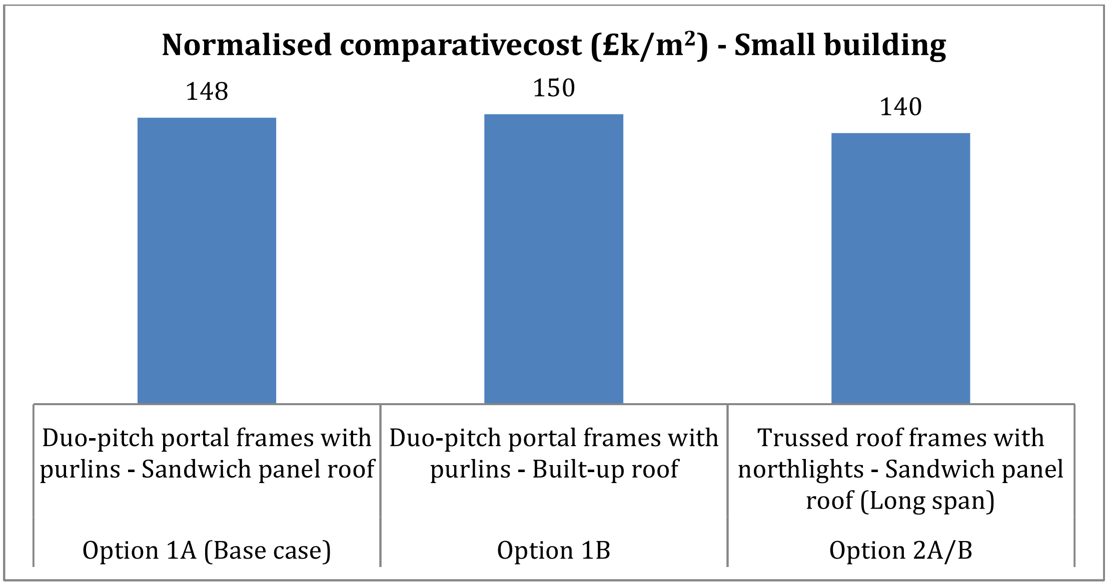


Figure 8.26 Normalised comparative costs – Small building

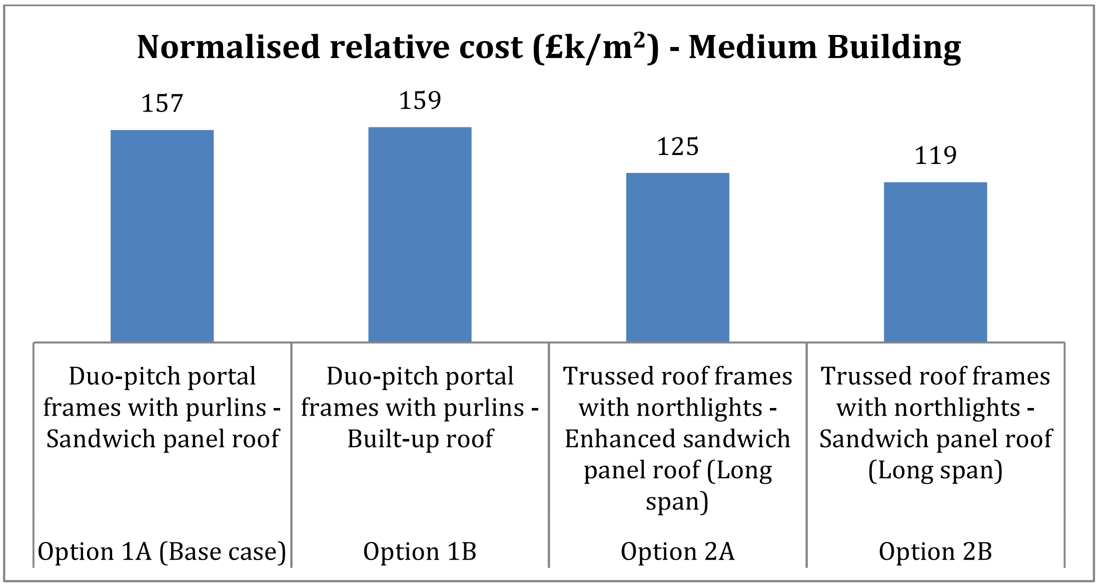


Figure 8.27 Normalised comparative costs – Medium building

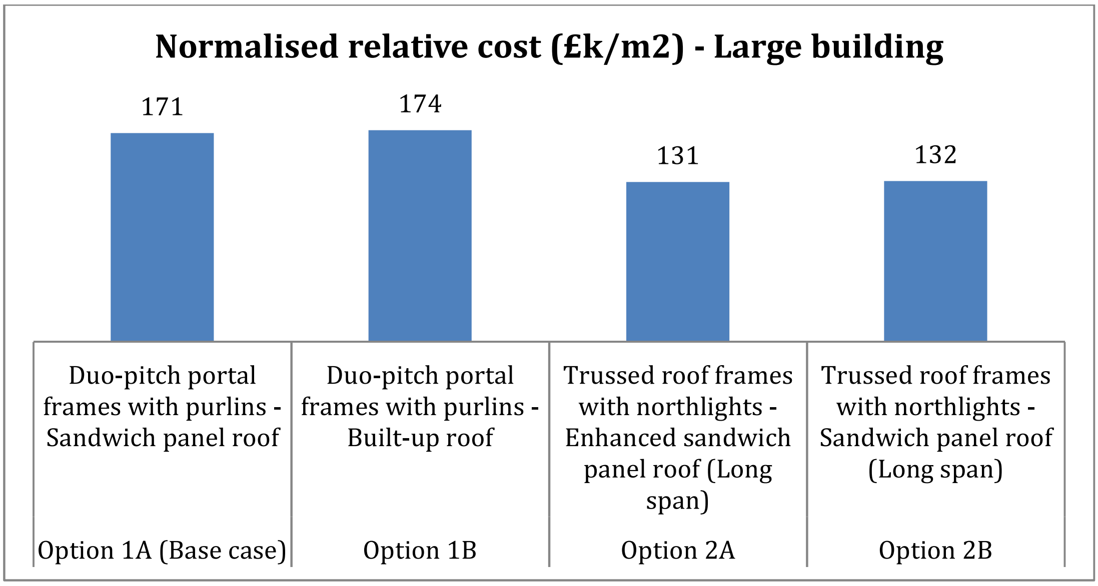


Figure 8.28 Normalised comparative costs – Large building

8.4.2.2 Cost breakdown

The total costs and costs breakdown per component type and activity are shown in Figure 8.29, Figure 8.30 and Figure 8.31 for each option and building size. The following key observations may be drawn from these figures:

- The roof and wall cladding (material and manufacturing) have the greatest cost contribution among all elements. The highest roof envelope cost is for Option 2 where a larger roof area needs to be covered. Option 2A is more expensive than Option 2B due to the use of an enhanced sandwich panel system, which is more costly. Option 1B was found to be the cheapest in terms of roof cladding. Wall cladding costs were quite consistent across the schemes. Small variations could only be noticed for Option 2 where a slightly larger wall area was required to be covered.
- The highest costs associated with the structure were for the primary and secondary components fabrication and materials, while erection showed a smaller contribution (11% for portal frames, 7%-8% for trusses). It should be highlighted that the cost of the materials and manufacturing of purlins in Option 1 is significant within the total costs associated with steelwork (25%-36%). Option 1 has higher material and erection costs than Option 2, due to the higher steelwork weight and also due to the significant contribution of the high purlin costs. Option 2 has higher fabrication costs per tonne, due to the truss structure and also due to use of tubular components which are more expensive in terms of fabrication. However, this is cancelled out by the reduced frame weight, yielding

an overall lower fabrication cost for the medium and large building sizes. For the small building size, the fabrication cost of Option 2 is slightly higher.

- The effects of the superstructure's weight on the foundations are also apparent. Option 1 demands heavier, hence more costly, foundations due to the increased frame weight. Option 1B requires heavier foundations than Option 1A since the weight of the roof cladding is increased with the mineral wool insulation, which is much heavier than PIR. However, foundations were found to have a small cost contribution overall.
- The cost of rooflights, northlights and flashing was found to have a very small contribution to the total cost, with little difference shown among the various options.

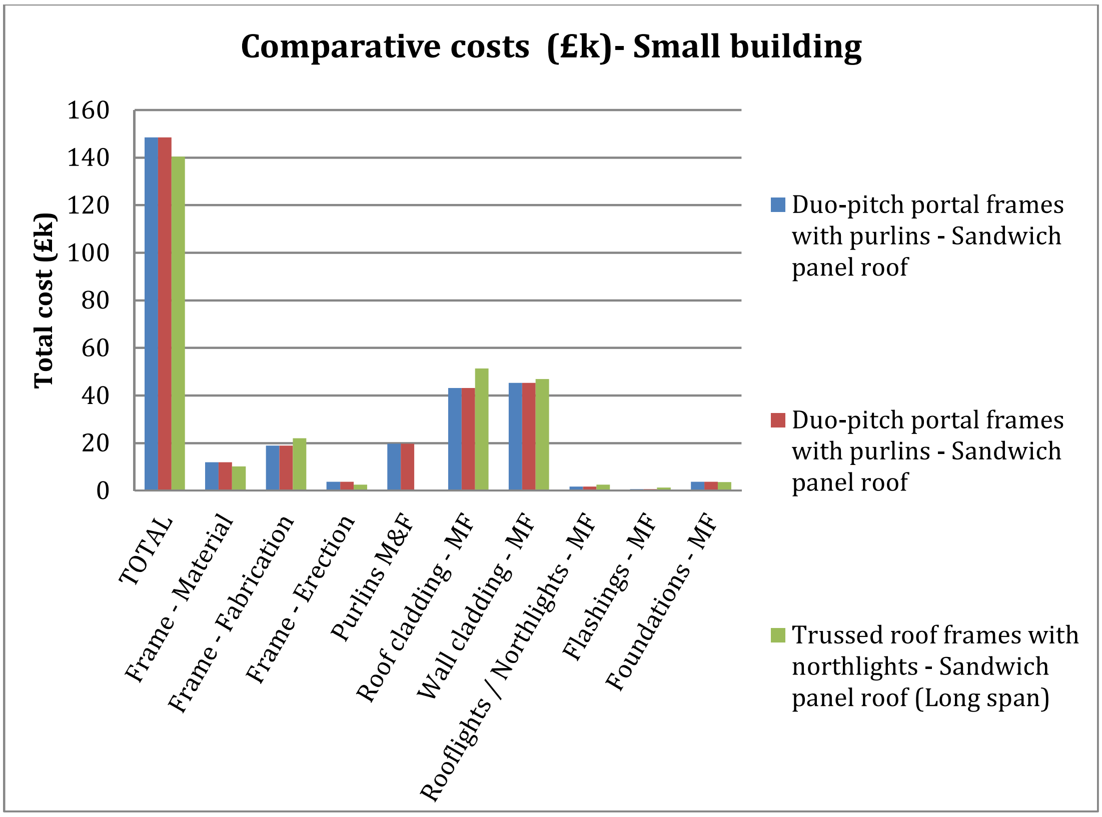


Figure 8.29 Comparative costs – Small building

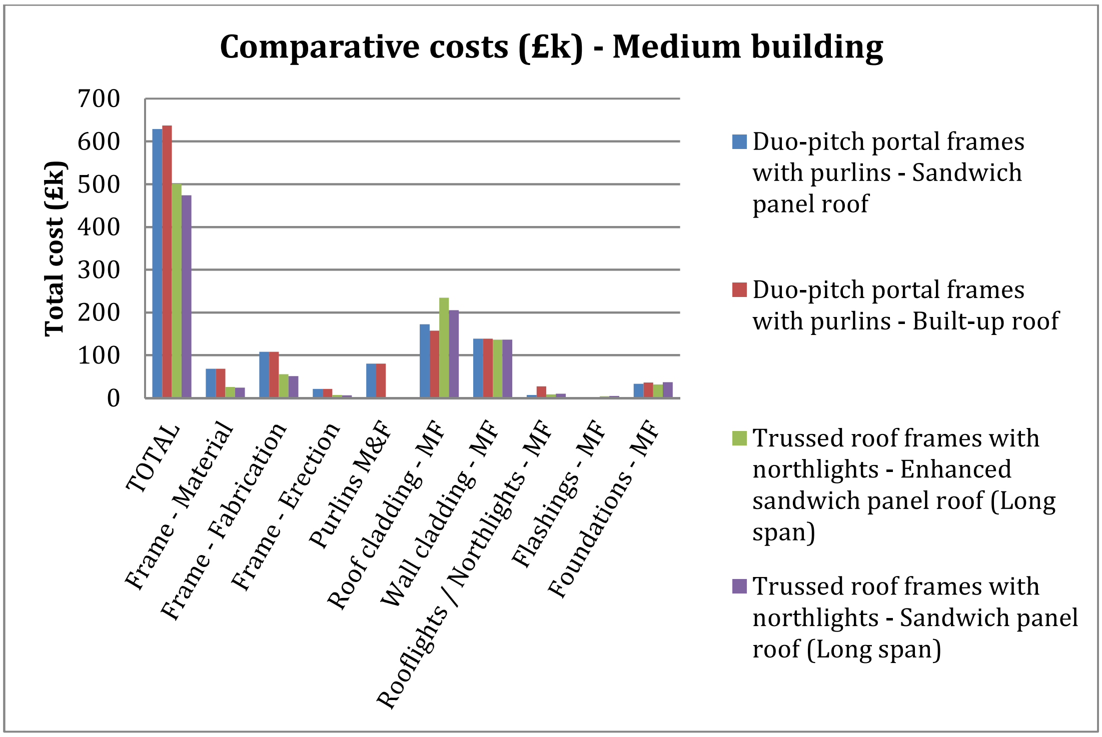


Figure 8.30 Comparative costs – Medium building

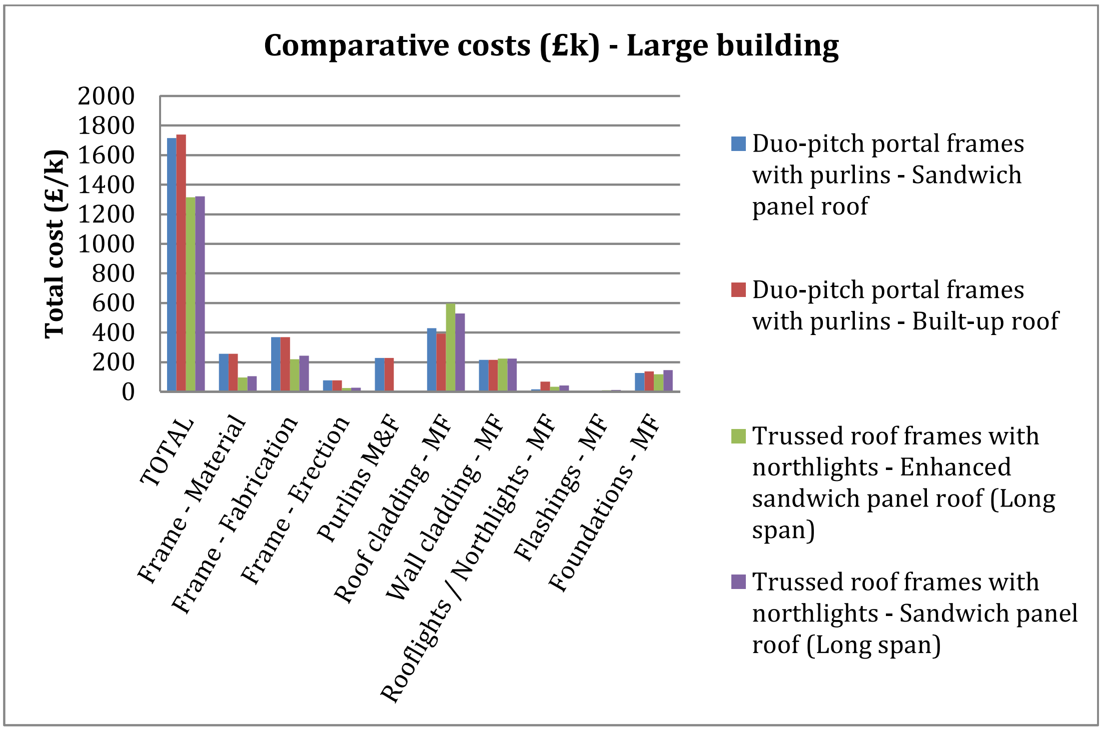


Figure 8.31 Comparative costs – Large building

Figure 8.32 to Figure 8.42 show the cost distribution percentages and the relative significance of different component types and activities within every building option and size. The following key observations were drawn:

- For the portal frame option, the frame (primary and secondary steelwork material, manufacturing and erection) accounts for 35%-52% of the total cost, with a higher percentage as the building size increases. Purlins account for 12%-13% of the total cost.
- For the trussed roof frame options, the frame accounts for 17%-28% of the total cost. This reduced percentage is associated with the reduced frame weight. While the fabrication of trusses shows higher costs compared to the portal frames, the reduced materials and erection costs for the reduced tonnage are much lower.
- Roof cladding accounts for 26%-32% for the portal frame options and 37%-47% for the long span options. This higher percentage in Option 2 is because of the lower frame contribution within the total costs and also due to the increased roof area and the use of enhanced, hence more expensive, sandwich panels for Option 2A in medium and large building sizes.
- The contribution of wall cladding varied between 29%-33% for the small building, 21%-29% for the medium building and 12%-17% for the large building. The percentage was higher when moving to Option 2 due to the slightly larger wall area that needed to be covered. The relative cost importance of wall cladding reduces as the building sizes increases due to the increased relative importance of the other elements.
- The contribution of foundations varied between 2%-3% for the small building, 5%-8% for the medium building and 8%-11% for the large building. The percentage was a higher percentage when moving to Option 2 due to the decreased participation of the frame. The relative cost-importance of the foundations increase as the building sizes increases due to the higher total weight of the building.
- The rooflights and northlights contribution to the whole cost was very small, in the range of 1-4%, with the GRP systems for built-up roofs showing the highest percentages.

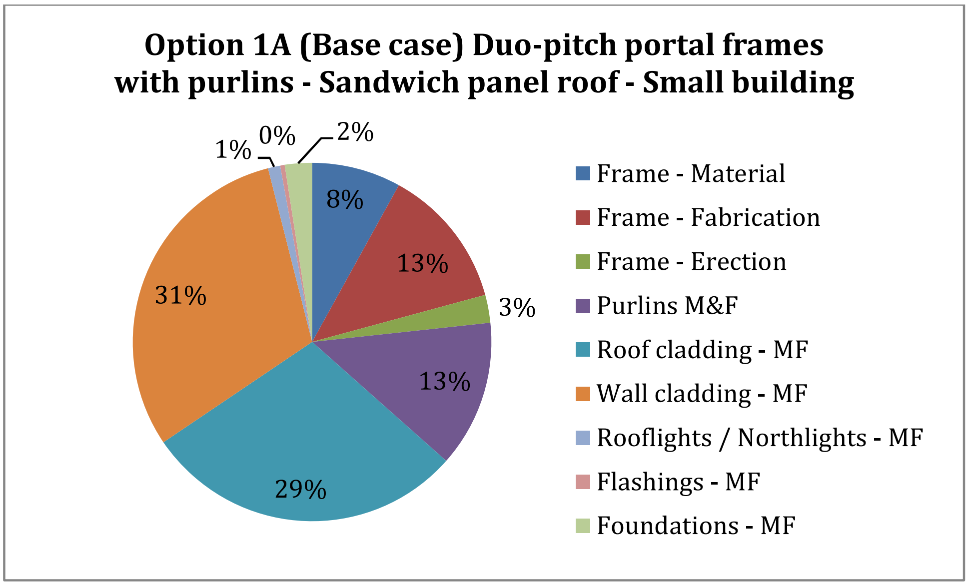


Figure 8.32 Cost distribution – Option 1A – Small building

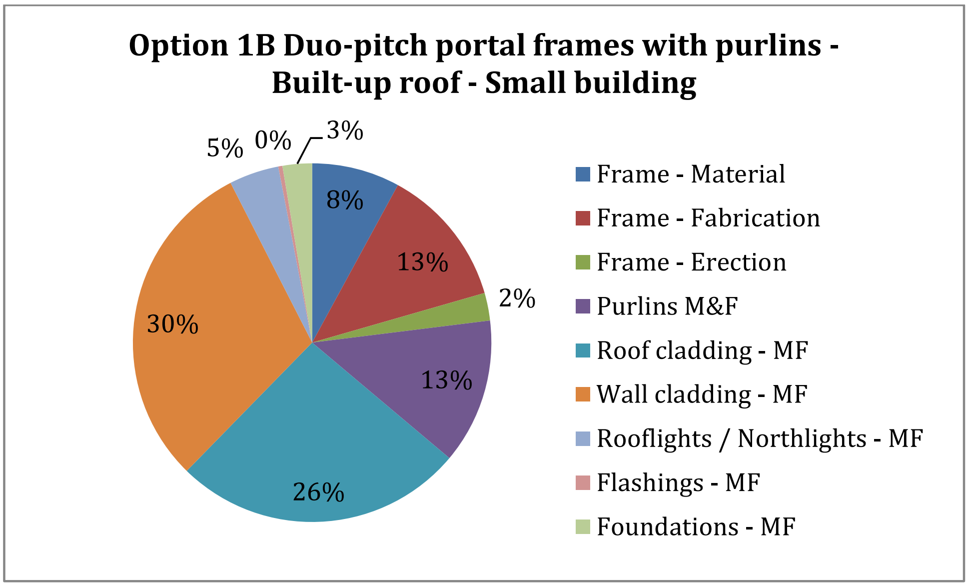


Figure 8.33 Cost distribution – Option 2B – Small building

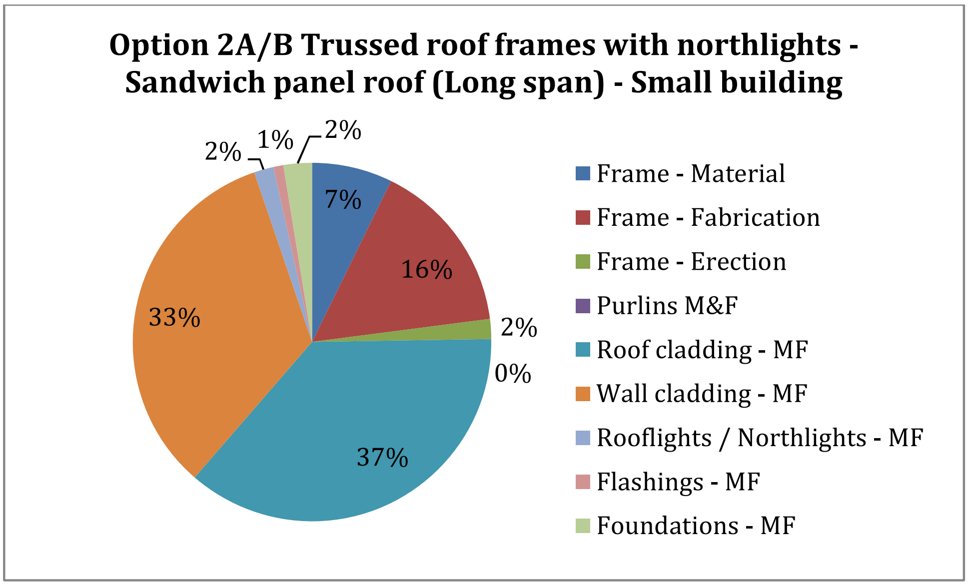


Figure 8.34 Cost distribution – Option 2A/B – Small building

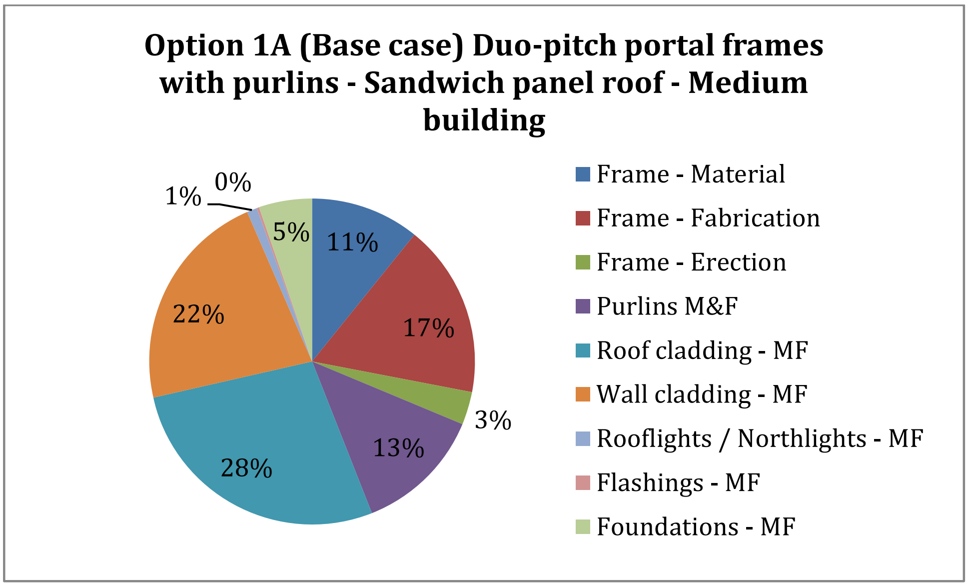


Figure 8.35 Cost distribution – Option 1A – Medium building

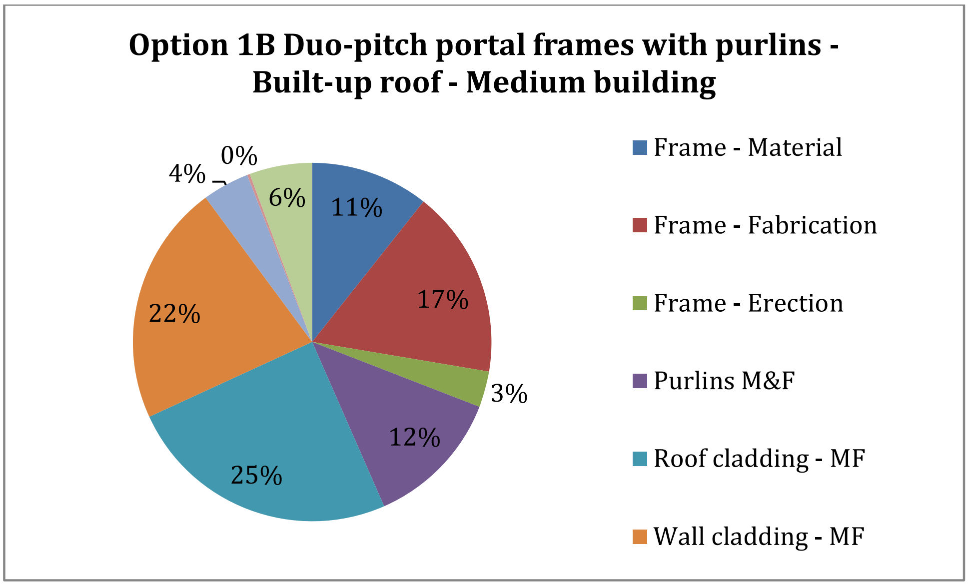


Figure 8.36 Cost distribution – Option 2B – Medium building

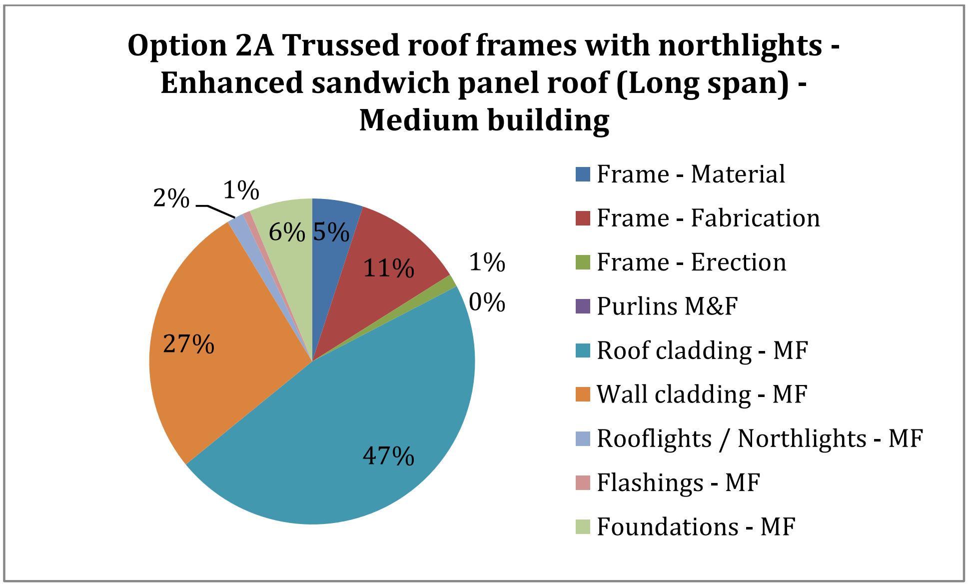


Figure 8.37 Cost distribution – Option 2A – Medium building

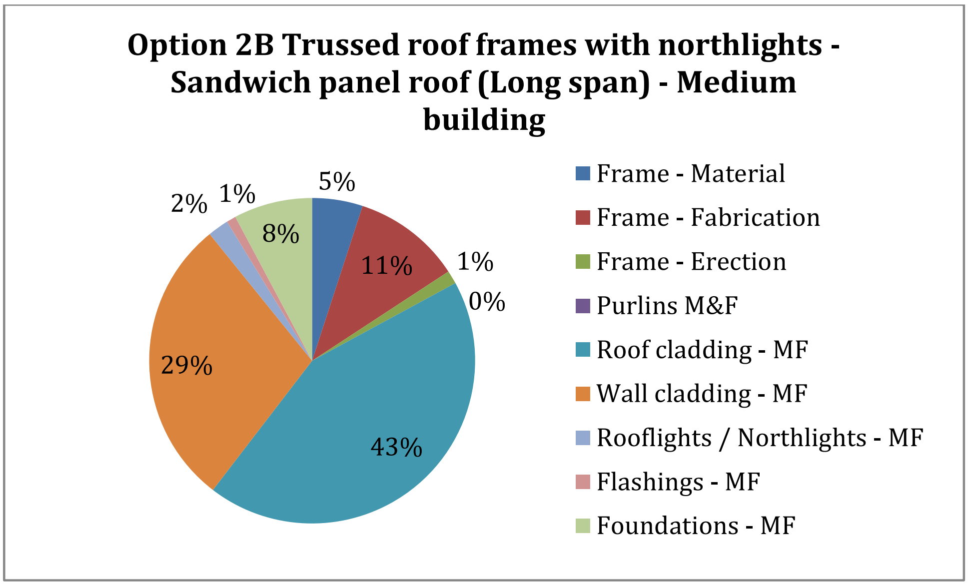


Figure 8.38 Cost distribution – Option 2B – Medium building

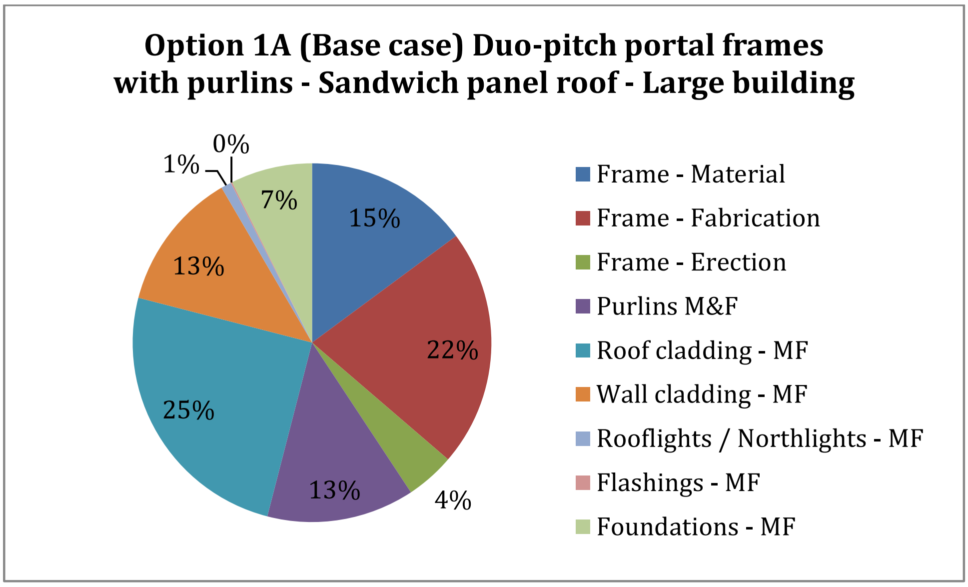


Figure 8.39 Cost distribution – Option 1A – Large building

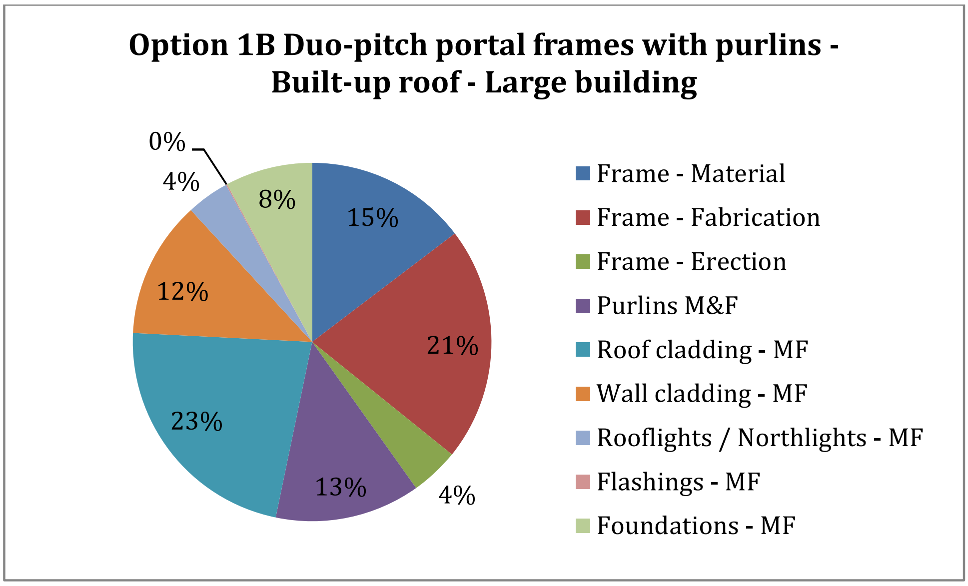


Figure 8.40 Cost distribution – Option 1B – Large building

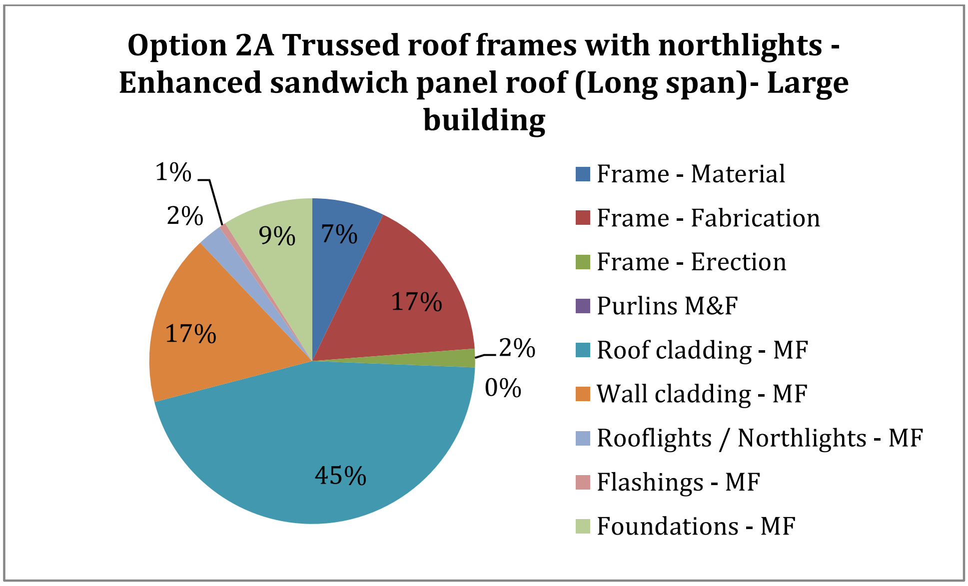


Figure 8.41 Cost distribution – Option 2A – Large building

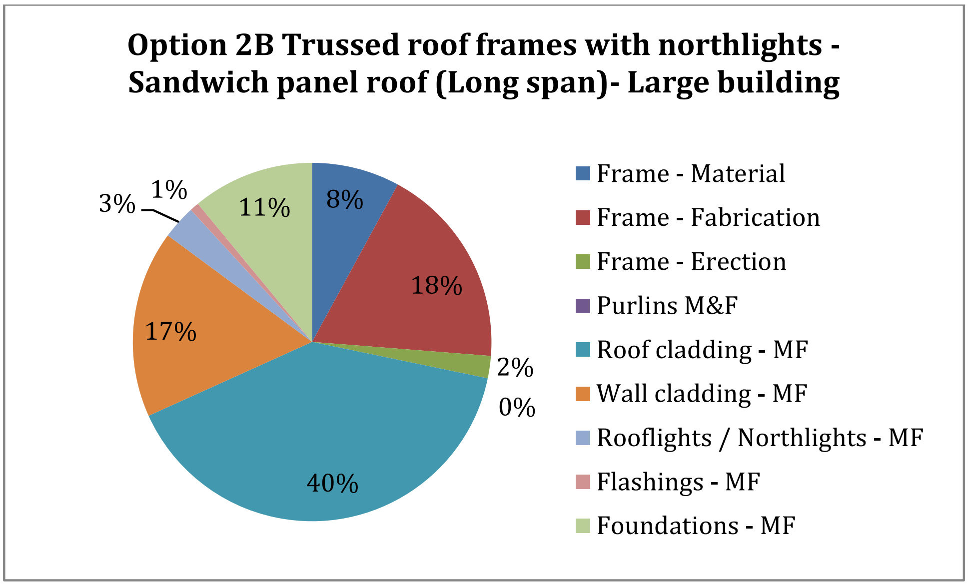


Figure 8.42 Cost distribution – Option 2B – Large building

8.4.2.3 Impact of northlights glazing type

Figure 8.43, Figure 8.44 and Figure 8.45 show the total comparative costs normalized per unit floor area when the glass option instead of the polycarbonate is used for the northlights. A comparison with Figure 8.26, Figure 8.27 and Figure 8.28 shows that a considerable increase of the total cost occurs for the glass option (16%-28%, higher percentage as building size increases). This is because the glass system is much more expensive compared to the polycarbonate. Furthermore, the glass option is much heavier than the polycarbonate which causes a negating effect on the foundations, required to be designed for higher load and, consequently, higher volume is needed.

Despite the change, Option 2 remains still the scheme with the lowest total comparative cost for medium and large buildings, although the benefit against Option 1A is now lower compared to the polycarbonate northlights option (-7.6%, -9.6% for medium building and -5.7% and -0.5% compared to the Option 1). Best options were Option 2B for the medium building and Option 2A for the large building. Exception is for the small building case, where Option 2 is more expensive than Option 1.

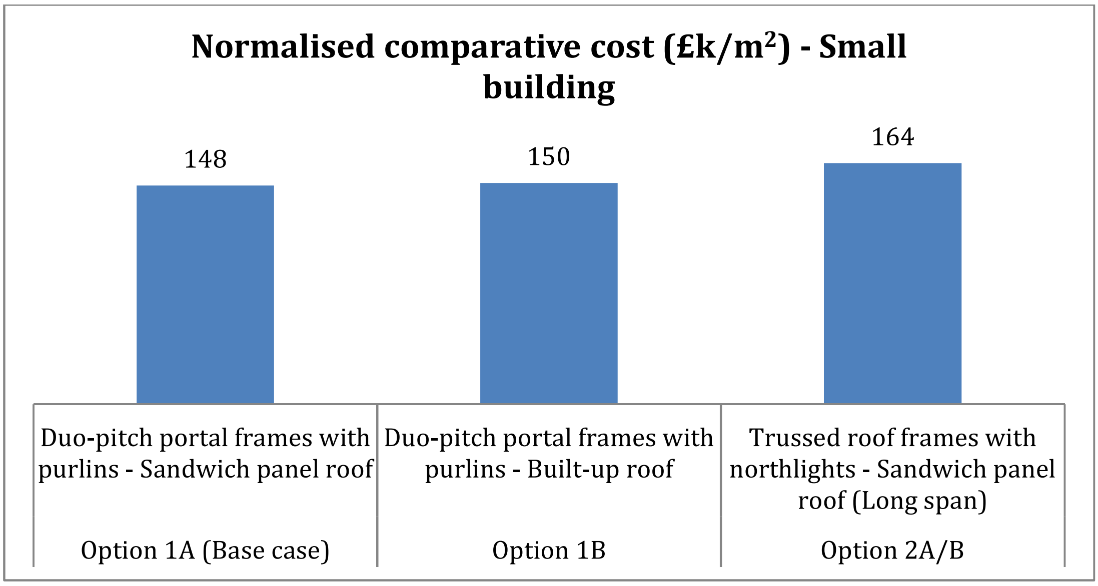


Figure 8.43 Normalised comparative costs – Small building with glass northlights

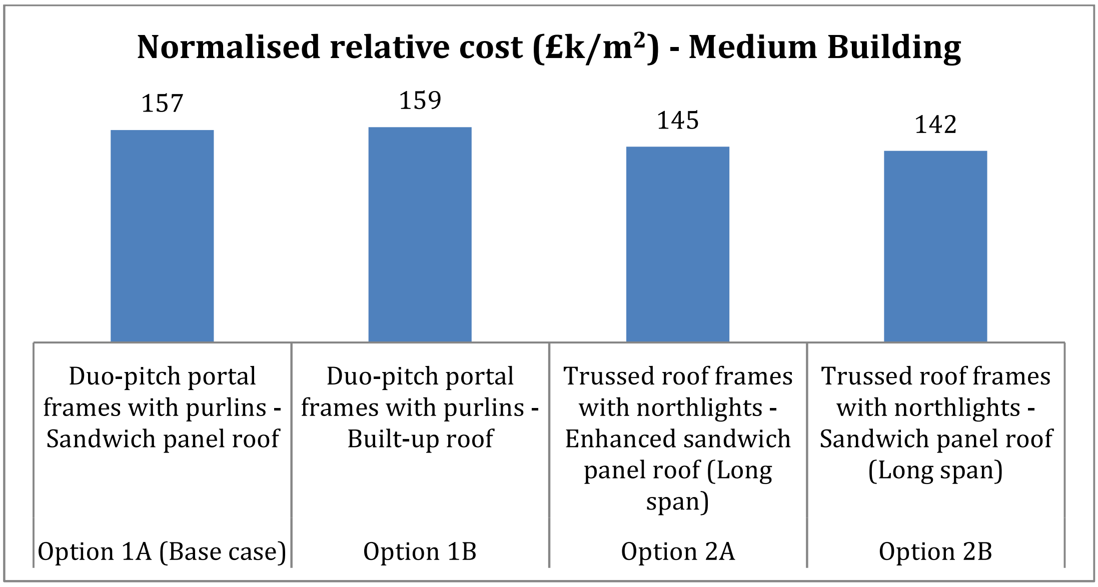


Figure 8.44 Normalised comparative costs – Medium building with glass northlights

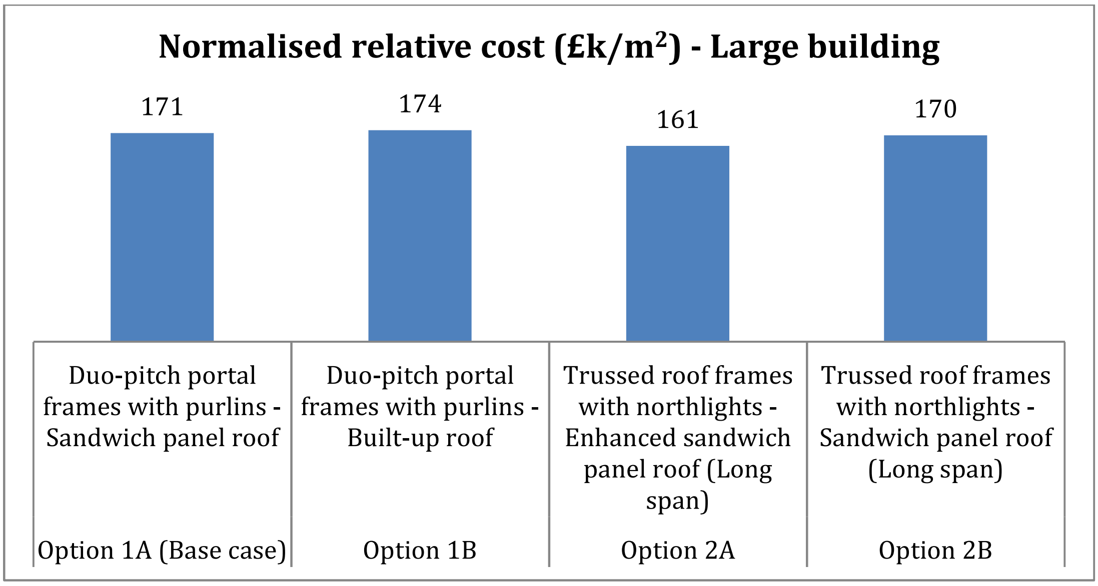


Figure 8.45 Normalised comparative costs – Large building with glass northlights

8.4.2.4 Impact of rooflights area

Figure 8.46, Figure 8.47 and Figure 8.48 show the variation of the total comparative cost normalized per unit floor area when the percentage area of rooflights changes from 15% to 20%. When the figures are compared to Figure 8.26, Figure 8.27 and Figure 8.28 it may be noticed that the cost of Option 1A decreases (1.1%-1.3%) while for Option 1B increases (<0.5%). This is because the cost of the polycarbonate system used for sandwich panel roofs is lower compared to the sandwich panel roof system per unit area, hence a part of the roof with higher impact is substituted with components of lower impact. For the built-up roof case, the cost of the GRP rooflight system is almost the same per unit roof area, hence the total difference is minimal. This observation is useful since the rooflights area may vary depending on the operational energy performance and optimisation requirements. Nevertheless, it is demonstrated that the variation is almost negligible.

There is obviously no impact on the trussed-roof frames since the area of northlights is dictated by the truss height. Also, the trussed-roof options show still lower total comparative cost.

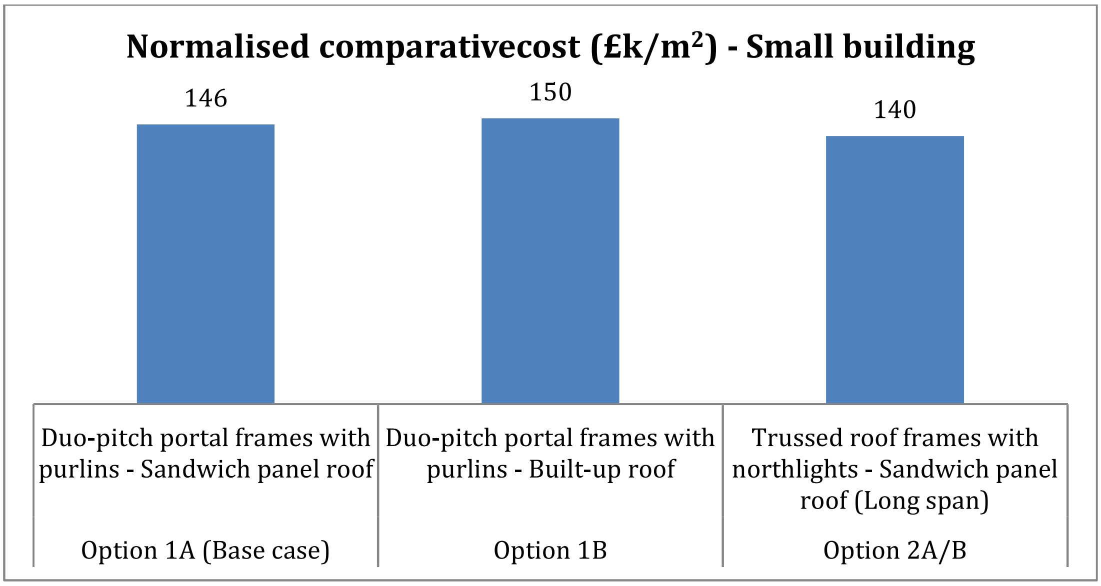


Figure 8.46 Normalised comparative cost – Small building with 20% rooflights

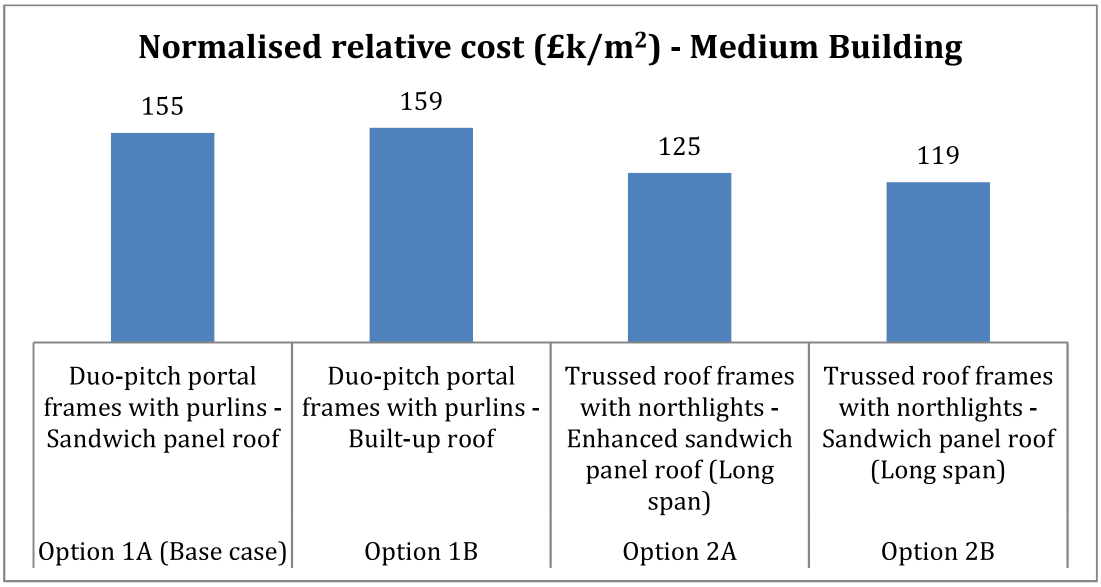


Figure 8.47 Normalised comparative cost – Medium building with 20% rooflights

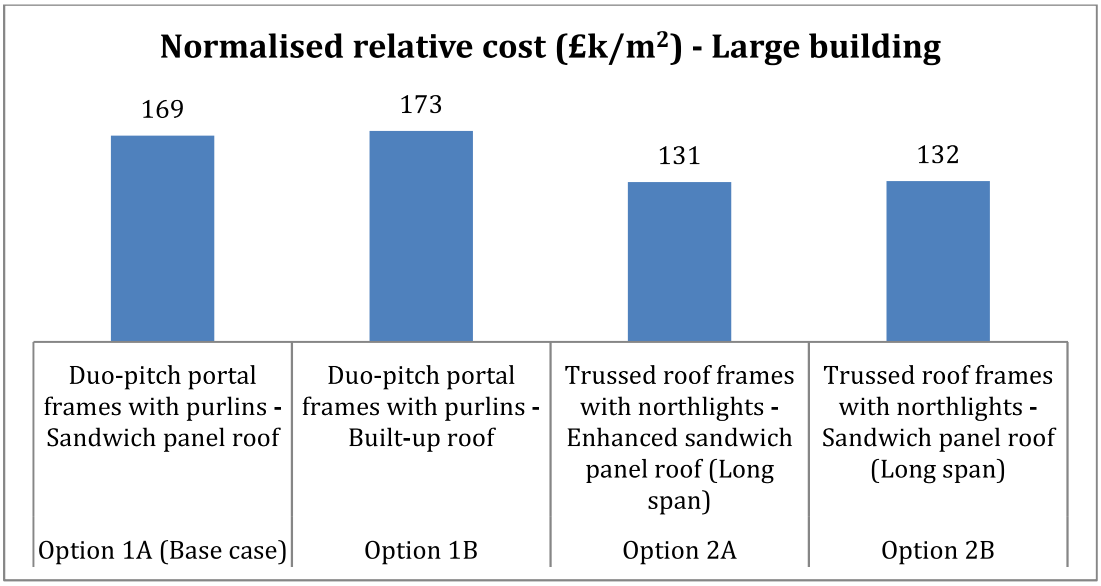


Figure 8.48 Normalised comparative cost – Large building with 20% rooflights

8.4.2.5 Impact of ground floor slab and sub-base

The cost impact of the ground floor slab and the sub-base are shown in Figure 8.49. The cost includes machine excavation, filling, membrane installation and pouring of concrete. The analysis shows that these costs are not negligible and they impose an additional £26k/m^2 for each building regardless the option and size.

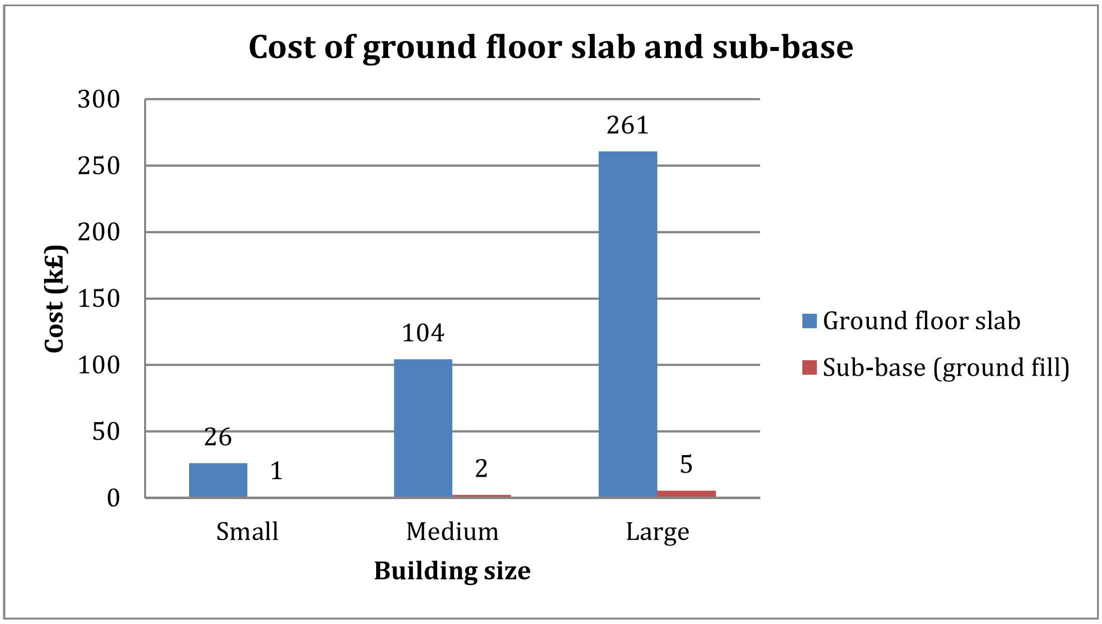


Figure 8.49 Cost impact of ground floor slab and sub-base

8.4.2.6 Comparison of parts count

A summary of the parts count for each option and building size is shown in Table 8.6, Table 8.7 and

Table 8.8. The following key observations were drawn:

- Option 2 has the significant benefit of eliminating the installation activities of the very large number of purlins, anti-sag rods and cleats, potentially reducing the construction time significantly.
- There is a slightly increased roof cladding area which needs to be covered for Option 2 compared to Option 1. However, the number of fastening rows required for the installation of long span components is reduced massively with Option 2 when compared to Option 1 where cladding should be fastened at each purlin. This is a significant benefit of long span envelope. Option 2A demanded fewer long span components (and roof area) than Option 2B due to the decreased frame number. Furthermore, the benefit of installing one layer of panel as single manufactured unit remains considerable when compared against built-up system installation.
- The elimination of purlins and reduced number of fastenings for roof cladding also reduces the time of working at height, which is a significant health and safety benefit.
- Sandwich panels installed as single manufactured units (Option 1A and Option 2) are much less labour intensive than Option 1B where a multi-layered with built-up system with individual layers of sheeting, insulation and bar and bracket components are required to be installed.
- There were the same number of rafters and main trusses for Options 1 and 2A, however, but Option 2A required additional edge trusses. Option 2B demands an increased number of main and edge trusses due to the higher number of frames.
- The number of columns was the same among Options 1A, 1B and 2A, while more columns are required for Option 2B.
- Options 1A, 1B, 2A demanded the same amount of base connections, while Option 2B required more due to more frames in the building. For the small building there was no difference since Option 2A and 2B coincide. It was also noted that the truss options require fixed bases, rather pinned for portal frames, which are more labour intensive.
- Option 2 required no apex connections as Option 1; however there is a significantly higher number of joints for the main truss segments and at the

main-edge truss locations. On the other hand, Option 1 requires a considerable number of eaves strut connections which are not required by Option 2, although substituted by a higher number of edge truss joints.

- The flashings' lengths for Option 2 were approximately 2.5-3 times the length of Option 1. This is due to the increased length of interfaces formed by the northlights geometry. Furthermore, Option 2A has shorter flashings length than Option 2B due to the decreased number of frames.
- The northlights area to be covered in Option 2 is higher than the rooflights area in Option 1. Also, Option 2B demanded larger northlights area than Option 2A due to the higher number of frames within the building.

Table 8.6 Summary of part counts – Small building

Activity	Unit	Option 1A	Option 1B	Option 2A/B
Site – preparation	m^2	1000	1000	1000
Foundation footings	No.	14	14	14
Ground floor slab	m^2	1000	1000	1000
Columns	No.	14	14	14
Rafters	No.	14	14	0
Main trusses	No.	0	0	14
Connections at • **Base** • **Apex** • **Eaves** • **Eaves struts** • **Main truss joints** • **Edge truss joints**	 No. No. No. No. No.	 14 14 7 24 0 0	 14 14 7 24 0 0	 14 0 7 0 13x2=26 12x3=36
Purlins • **Cleats** • **Anti-sag rods**	No. No. No.	70 98 78	70 98 78	0 0 0
Eaves struts	No.	12	12	0
Edge trusses	No.	0	0	12
Roof bracings	No.	28	28	28
Wall bracings	No.	8	8	8
Roof sandwich panels (continuous)	m^2	855	0	0
Roof built-up system (continuous) • **Outer sheet** • **Liner sheet** • **Insulation** • **Bar** • **Bracket**	m^2	0	855	0
Long span roof panels	No. m^2	0 0	0 0	153 1020
Rows of cladding-structure fastenings	No.	2800	2800	306
Wall cladding	m^2	553	553	573
Rooflights	m^2	128	128	0
Northlights	m^2	0	0	197
Flashings	m	186	186	404

Table 8.7 Summary of part counts – Medium building

Activity	Unit	Option 1A	Option 1B	Option 2A	Option 2B
Site – preparation	m^2	4000	4000	4000	4000
Foundation footings	No.	22	22	22	26
Ground floor slab	m^2	4000	4000	4000	4000
Columns	No.	33	33	33	39
Rafters	No.	44	44	0	0
Main trusses	No.	0	0	33	39
Connections at • **Base** • **Apex** • **Eaves** • **Eaves struts** • **Main truss joints** • **Edge truss joints**	 No. No. No. No. No.	 22 22 22 40 0 0	 22 22 22 40 0 0	 22 0 22 0 32x2=64 20x3=60	 26 0 26 0 38x2=76 24x3=72
Purlins • **Cleats** • **Anti-sag rods**	No. No. No.	196 308 540	196 308 540	0 0 0	0 0 0
Eaves struts	No.	20	20	0	0
Edge trusses	No.	0	0	20	24
Roof bracings	No.	56	56	56	56
Wall bracings	No.	8	8	8	8
Roof sandwich panels (continuous)	m^2	3419	0	0	0
Roof built-up system (continuous) • **Outer sheet** • **Liner sheet** • **Insulation** • **Bar** • **Bracket**	m^2	0	3419	0	0
Long span roof panels	No. m^2	0 0	0 0	507 4054	612 4078
Rows of cladding-structure fastenings	No.	15680	15680	1014	1224
Wall cladding	m^2	1691	1691	1665	1665
Rooflights	m^2	513	513	0	0
Northlights	m^2	0	0	657	786
Flashings	m	445	445	1197	1403

Table 8.8 Summary of part counts – Large building

Activity	Unit	Option 1A	Option 1B	Option 2A	Option 2B
Site – preparation	m^2	10000	10000	10000	10000
Foundation footings	No.	32	32	32	40
Ground floor slab	m^2	10000	10000	10000	10000
Columns	No.	48	48	48	60
Rafters	No.	64	64	0	0
Main trusses	No.	0	0	48	60
Connections at • **Base** • **Apex** • **Eaves** • **Eaves struts** • **Main truss joints** • **Edge truss joints**	 No. No. No. No. No.	 32 32 32 60 0 0	 32 32 32 60 0 0	 32 0 32 0 47x2=94 30x3=90	 40 40 40 0 59x2=118 38x3=114
Purlins • **Cleats** • **Anti-sag rods**	No. No. No.	437 736 1350	437 736 1350	0 0 0	0 0 0
Eaves struts	No.	30	30	0	0
Edge trusses	No.	0	0	30	38
Roof bracings	No.	88	88	88	88
Wall bracings	No.	8	8	8	8
Roof sandwich panels (continuous)	m^2	8505	0	0	0
Roof built-up system (continuous) • **Outer sheet** • **Liner sheet** • **Insulation** • **Bar** • **Bracket**	m^2	0	8505	0	0
Long span roof panels	No. m^2	0 0	0 0	1238 10309	1596 10485
Rows of cladding-structure fastenings	No.	54625	54625	2476	3192
Wall cladding	m^2	2628	2628	2723	2722
Rooflights	m^2	1276	1201	0	0
Northlights	m^2	0	0	2523	3195
Flashings	m	685	685	2731	3392

8.5 Concluding remarks

A study was carried out to compare the options of traditional portal frame construction with sandwich panels roofs (Option 1A) or built-up roofs (Option 1B) against the trussed-roof frames with northlights comprising enhanced long span sandwich panels at optimum frame spacing (Option 2A) or conventional long span sandwich panels with frame spacing at maximum panel span distance (Option 2B). The comparison was carried out in terms of embodied carbon emissions, impact of structure on the operational carbon emissions and construction cost.

Only the elements which varied among the different options were included in the embodied carbon and cost appraisals and these are listed in Section 8.2.1 and 8.4.1 respectively.

The study concluded on the following key findings:

The use of trussed-roof frames with northlights and long span roof sandwich panels showed decreased embodied carbon compared to traditional portal frame construction with purlins and sandwich panel or built-up roof systems. Compared to tradition portal frame with sandwich panel roof cladding (Option 1A), the reduction was 1.8%, 5.0% and 5.3% for the small, medium and large buildings respectively. Compared to traditional portal frame with built-up roof cladding (Option 1B), the reduction was 7.2%, 11.4% and 11.5% for the small, medium and large buildings respectively. It was found that the concrete foundations were the greatest contributors of embodied carbon. Although the floor slab was not included in the comparative study, it was found to have almost the same embodied carbon impact as the whole superstructure and foundations together. PIR insulation and cladding in general were found to also have a significant impact depending on the option.

The trussed roof frame was also found to be less expensive by up to 10.0%, 26.6% and 26.8% for the small, medium and large buildings respectively. The use of glass northlights showed a total cost increase of 16%-28% against the polycarbonate glazing option, while the variation of rooflights showed minimal cost function.

The trussed-roof option also showed reduced part counts particularly in terms of purlins, anti-sag rods and cleats. Also, far lesser connections would be required for the installation of the long span roof cladding. These facts indicate increased speed of the metal roof envelope installation. On the other hand, there is some considerable increase in terms main frame joints (edge to main trusses, truss segments) and glazing area, and a severe increase in terms of flashings. There was little variation in the number of the pre-fabricated frame components.

The careful selection of northlights height and area, frame spacing and glazing type was considered to be essential to define optimum operational carbon performance. Based on earlier literature, the northlights construction would demand less lighting and cooling energy and higher heating energy compared to the rooflights option in an annual level. The operational energy demand for industrial buildings is vastly dominated from lighting. Also, it is anticipated that the energy demand for cooling will increase and for heating will decrease in the following decades due to climate change. All these features make the northlights construction an appealing option.

While significant effort is dedicated in reducing the embodied carbon in the structure and mainly in the steel frames, it was shown that a meaningful carbon reduction can occur if the very high impact of the reinforced concrete ground floor slab can be challenged. Alternative construction methods for ground floor slab in industrial buildings, such as use of thin slabs on non-bonded materials can be further investigated and could potentially have a significant contribution towards reducing the embodied carbon of the building.

Furthermore, the PIR insulation has a significant impact on the embodied carbon of the superstructure, due to its high emissions rate per unit weight (almost 3 times of steel's). Although this could shift the argument towards more use of mineral wool, the increased weight of the latter, especially for structural insulation, and its impact on the steel frame and foundations design should not be disregarded. Alternative materials of structural insulation, such as polysterene, have much lower emissions rate while possessing similar mechanical properties as the PIR. Development or use of structural insulation materials with low emissions, good thermal and fire resistance properties for engineering of long span roof envelope systems is probable to yield high carbon savings for the superstructure.

Chapter 9 Conclusions and recommendations for further research

The current chapter presents the conclusions with reference to the aim and objectives presented in Chapter 1 and recommendations for further research.

9.1 Background review

The trend of the UK Building Regulations to date has been to specify continuously increasing insulation thicknesses for the building envelope to minimise energy losses, resulting in higher levels of embodied carbon for the building as a result of increased material usage. However, the structural capability of the envelope that arises from the thicker insulation is not currently fully exploited. Highly insulated building envelopes possess structural capability in terms of strength and stiffness that could be utilised to minimise materials usage and reduce materials related embodied carbon. Therefore, an opportunity exists to maximise the utility of the envelope to reduce the overall amount of material and consequential embodied carbon within the whole building.

The increase in structural capacity due to thicker insulation is particularly apparent in sandwich panels, where the strength of the panel relies on the composite action between the insulation core and bonded metal faces. The increase of the insulation depth leads to an increase of the component’s stiffness and resistance. This presents an opportunity to make greater structural use of the envelope, permitting the removal of some structural elements and reducing the overall level of embodied carbon within the building.

The focus of the present study is on single storey industrial buildings, since almost 60% of the constructional steelwork in the UK is used in this sector. With steel portal frames dominating the single-storey frame market in the UK, any savings in this type of building will have a significant impact in this market sector. Whilst portal frame construction provides optimised economy and efficiency for the structure itself, it is not necessarily the optimal form for the building as a whole. Alternative structural forms may yield greater benefit when viewed holistically, especially when the structural capability of the envelope is utilised.

9.2 Accomplishment of aims and objectives

As discussed in Chapter 1, the primary aims of this research were to:

- *Quantify the benefit associated with reduction of structure within the building by exploiting the increased insulation depth and structural capability of sandwich panel envelope systems.*
- *Address the technical barriers to the implementation of more efficient structure-envelope assemblies.*

An extensive literature search ***(Objective 1)*** was initially undertaken and is presented in Chapter 2. This search was undertaken to (a) review the technology and state-of-the-art in the design and construction of single storey industrial buildings in the UK; (b) review the UK regulatory context for carbon emissions, the trend of the future likely energy conservation requirements, the environmental performance of single-storey industrial buildings, the role of the building envelope and the related opportunities for embodied carbon emissions reduction; (c) identify opportunities for exploiting the structural capabilities of sandwich panels arising from the increased insulation thickness in single story buildings; and (d) review earlier work and state-of-the-art associated with these opportunities.

Based on the output of the literature review, a methodology to address the aims was developed and presented in detail in Chapter 3.

9.2.1 Feasibility studies

A set of feasibility studies was initially undertaken to evaluate the opportunities for exploiting the structural capability of sandwich panels to achieve reduction of frame material. The feasibility studies referred to the following objectives:

(Objective 2) *Determine the opportunities for exploiting the structural capability of sandwich panel envelope systems in terms of:*

- *Increasing the span of the cladding elements (reducing the number of structural members).*
- *Utilising diaphragm action within the envelope (building stability and stiffness).*
- *Removal of primary frame elements (frameless construction for small buildings).*

(Objective 3) *Determine the structural forms that are best able to utilise the structural capability of sandwich panels, identify barriers (technical and commercial) to the uptake of these types of structure and propose solutions to overcome these barriers.*

(Objective 5a) *Review the envelope – structural forms assemblies in terms of:*

- *Structural efficiency, based on structural weight.*

Three typical warehouse building sizes (small, medium, and large) were chosen for the long span and diaphragm action studies, considered typical and sufficiently representative in the UK. For the frameless buildings study, the building sizes were limited by the spanning capability of modern roof sandwich panel systems.

The key findings of the feasibility study for each opportunity are listed below:

9.2.1.1 Buildings with long span roof envelope systems

Four structural frame options were chosen to be examined for the long-span opportunity:

1. Duo-pitch portal frames with purlins (base case)
2. Duo-pitch portal frames without purlins
3. Flat-pitch multi-bay re-oriented portal frames
4. Frames with trussed roof system and north lights

The following key findings were drawn from the study:

- The trussed roof frames with north lights provided the best option for significant steelwork reduction (38%-60% against base case) while simultaneously offering large clear spans and a good solution for energy conservation through natural lighting. Their benefits and feasibility were evident for the whole range of building sizes.
- The re-oriented portal frames also showed significant steelwork weight reduction (15%-53% against base case), mainly for medium and large building sizes. However, this structural arrangement results in smaller clear spans between columns. This is a significant compromise to building owners' requirement for maximum unobstructed clear space to maximise the versatility of the available floor area, particular for medium and large building sizes.
- Duo-pitched portal frames without purlins showed limited steelwork weight reduction (6%-19% against base case), while they would require an extra layer

of cladding to allow for rainwater flow. Steelwork savings due to the elimination of purlins may be partly outweighed by the need for an additional cladding layer.

The trussed roof frames with northlights were found to yield a significant benefit in terms of material reduction for all building sizes, without compromises in achieving clear spans whilst potentially offering further energy conservation benefits. Moreover, the scheme and its identified technical barriers to be addressed suggest a small step change from the current practice. Therefore, it was decided that the trussed roof frames with northlights option would be taken forward for further research.

The optimum frame spacing was found to be 6.67m for the small building size and 8.00m for the medium and large building sizes. Roof sandwich panels available today can achieve 6.67m clear span without structural modifications, particularly suiting the spanning requirements identified for small building sizes. For medium and large buildings, improved spans, 8.00m, would be required to be achieved. Hence, further research would be required to specify improved roof sandwich panels to span at 8m. Such panels would be likely to possess higher embodied carbon due to increased material usage and, consequently, embodied carbon increases would also require to be quantified.

9.2.1.2 Diaphragm action

The opportunity to exploit the diaphragm action of the cladding was examined for the following four structural frame schemes:

1. Duo-pitch portal frames with purlins:
   - with normal (6^{o}) roof pitch (base case)
   - with high (12^{o}) roof pitch
2. Duo-pitch portal frames without purlins and long-span roof envelope spanning between rafters:
   - with normal (6^{o}) roof pitch
   - with high (12^{o}) roof pitch

The panel arrangements were modelled to span either between purlins or rafters to suit Schemes 1 and 2 respectively. Different fastening arrangements were investigated including combinations of (a) normal and dense fastening and (b) fastening on two or four sides of the shear panels. The case of 2-sided fastened panels with normal fastener spacing correspond to a typical 'as-built' cladding installation without diaphragm action provisions. 4-sided arrangements and dense fastening correspond to structurally 'enhanced' cases.

The study identified that there is no meaningful advantage for structural material reduction with the aid of diaphragm action. Steelwork weight savings were found to be up to 9.6% and 14.5% for the small and medium size buildings respectively. Savings were more obvious for buildings with high roof pitch portal frames and 'enhanced' envelope arrangements spanning directly between rafters, rather than purlins. No real steelwork reduction was found for 'as-built' envelope arrangements of low roof pitch.

Apart the limited steelwork savings, significant barriers are associated with advancing this opportunity. Importantly, high roof pitch buildings are highly unlikely to be implemented in practice due to increase of the building volume and consequential onerous requirements for operational energy. In addition, several technical barriers would require further research if the opportunity was to be advanced, primarily including: onerous effects of openings; strength improvements for 4-sided diaphragms due to premature failure as a result of attracting higher load; effects of combined in-plane shear and out-of-plane loads on panels; and overcoming onerous code requirements for frame similarity across the building.

Overall, there is no meaningful scope for frame material reduction with the aid of diaphragm action, whilst any small benefits are very limited compared to the long span opportunity. Hence, it was decided that the diaphragm action opportunity would not be taken forward for further research.

9.2.1.3 Frameless buildings

The opportunity of frameless building with the aid of sandwich panels was investigated on the basis of the panels replacing the conventional structural members and being used as part of the load bearing frame without a primary substructure. This would necessitate:

- Wall components resisting the lateral out-of-plane forces and, additionally, the vertical forces (acting as columns) and the in-plane lateral forces (acting as wall diaphragms for strength and stiffness).
- Roof components resisting lateral out-of-plane forces and, additionally, the in-plane lateral forces (acting as roof diaphragms for strength and stiffness).
- The frame stability relying on the rotational capacity of the wall/roof junctions and the flexibility of the base supports.

Feasibility studies were undertaken to examine the resistance and stability of:

- Roof systems for out-of-plane and in-plane shear loading
- Wall systems for out-of-plane, in-plane vertical and shear loading

The study showed that frameless buildings constructed from sandwich panels are feasible, hence there is scope for significant steelwork elimination. However, their size would be highly limited by the spanning capability of the roof panels. With the current sandwich panel technology, frameless design could only be applicable for very small buildings with roof spans up to 13.2m, which significantly limits the range of applications, whilst being a significant step change from the current practice.

Moreover, challenging technical barriers would require a significant degree of further research if the opportunity was to be advanced. These include primarily: challenging connection detailing, considering load introduction effects, connection flexibilities and thermal bridging; provisions and onerous effects of openings on wall / roof diaphragms and implications on global building stability; and significant work scope for experimental validation as current guidance is not fully validated by testing.

Overall, the frameless buildings opportunity is very limited, particularly when compared to the long span one, where the range of applications is wider and the scope for steelwork reduction is significant with only small improvements to the current practice. Hence, it was decided that the frameless opportunity would not be taken forward for further research.

9.2.2 Optimisation of long span sandwich panels

The feasibility studies concluded that trussed roof frames with north lights provide the greatest potential for steelwork reduction for an optimum frame spacing of 6.67m for small buildings and 8.00m for medium and large buildings. As typical roof sandwich panels currently available can already span 6.67m, the focus of the subsequent study was the re-engineering of the panels to achieve the 8.0m span required for the medium and large buildings.

In order to achieve the optimum span between adjacent frames, the structural performance of the sandwich panels needed to be improved. A subsequent study was undertaken to develop revised specifications for long span roof sandwich panels to increase their spanning capability, while minimising the increase in embodied carbon. The panels were considered to comprise one fully and one lightly profiled steel sheeting and PIR insulation cores, as typically for roof panels. The mechanical resistance of the panels was evaluated using a combination of theoretical analysis and structural testing and a Pareto-optimal set of solutions was found.

A structural testing programme was initially devised to determine (a) the effect of PIR core density on its mechanical properties and (b) the compressive resistance of fully and lightly profiled steel sheets with varying geometries. Linear relationships between the PIR core density and its mechanical properties were established based on the test data and some statistical post-processing. The test results to determine the compressive resistance of the steel sheets were compared against those obtained from existing analytical methods and the most appropriate method was selected.

The optimisation problem was defined as maximising the resistance of the panel while minimising the embodied carbon for the selected span distance (8.0m). A variable vector was defined, including all the geometrical and mechanical panel properties which influence the structural behaviour. The core's mechanical properties were related to its density. Design and manufacturing constraints were applied to the problem.

A set of solutions were derived, corresponding to sets of panel properties (as in the variable vector). For the applied loads, the Pareto-optimal panel solution and its corresponding embodied carbon were identified. Achievement of the optimal solution requires only minor modifications to the current panel properties. The increase of insulation depth was the dominant parameter to achieve a carbon-optimal panel at the increased span, followed by the height of the outer profile. Increasing the density of the core, steel thicknesses or profile geometries were found no effect in achieving optimality. The optimal panel design demonstrated 13.9% more embodied carbon compared to a currently available panel with similar thermal performance.

The study presented an enhanced panel specification which meets the requirement for roof sandwich panels spanning 8.0m. Also, it demonstrated that in order to take full advantage of identified frame material savings, only some minor changes to the panels are required to optimise their performance. These resulted in only a small embodied carbon increase for the panel.

9.2.3 Systems review

Having concluded that the long span opportunity provides the best promise for meaningful material reduction and having proposed sandwich panel solutions to accommodate enhanced sandwich panel requirements in terms of designing optimised long span sandwich panels, the study proceeded with the whole systems review as defined in the following objective:

(Objective 5b and 5c) *Review the envelope – structural forms assemblies in terms of:*

- *Embodied carbon, based on established databases and system boundaries, reflecting the identified envelope forms and considering the optimum combination of envelope and structure for the chosen buildings.*
- *Cost on site, based on calculated component and construction rates.*

A study was carried out to compare the options of traditional portal frame construction with sandwich panels roofs or built-up roofs against the trussed-roof frames with northlights comprising enhanced long span sandwich panels at optimum frame spacing or conventional long span sandwich panels with frame spacing at maximum panel span distance. The comparative study was made in terms of embodied carbon, impact of structure on the operational carbon and construction cost. Only the elements which varied among the different options were included in the embodied carbon and cost appraisals.

The study concluded that the use of trussed-roof frames with northlights and long span roof sandwich panels showed decreased embodied carbon compared to traditional portal frame construction with purlins and sandwich panel or built-up roof systems. Compared to traditional construction with sandwich panel roofs, the reduction was 1.8%, 5.0% and 5.3% for the small, medium and large buildings respectively. Compared to traditional portal frames with built-up roof cladding (Option 1B), the reduction was 7.2%, 11.4% and 11.5% for the small, medium and large buildings respectively. It was found that the concrete foundations were the greatest contributors of embodied carbon. Although the floor slab was not included in the comparative study, it was found to have almost the same embodied carbon impact as the whole superstructure and foundations together. PIR insulation and cladding in general were found to also have a significant impact depending on the option, while the choice of the rooflights or northlights glazing materials was found to have an important contribution as well.

The trussed roof frame was also found to be less expensive by up to 10.0%, 26.6% and 26.8% for the small, medium and large buildings respectively. The use of glass northlights showed a total cost increase of 16%-28% against the polycarbonate glazing option, while the variation of rooflights percentage showed minimal cost difference.

The trussed-roof option also showed reduced part counts particularly in terms of purlins, anti-sag rods and cleats. Also, fewer connections would be required for the installation of the long span roof cladding. These facts indicate increased speed of the metal roof envelope installation. On the other hand, there was some considerable increase in terms of main frame joints (edge to main trusses, truss segments) and glazing area, and a severe increase in terms of flashings. There was little variation in the number of the pre-fabricated frame components.

The careful selection of northlights height and area, frame spacing and glazing type was considered to be essential to define optimum operational carbon performance. Based on earlier literature, the northlights construction would demand less lighting and cooling energy and higher heating energy compared to the rooflights option. The operational energy demand for industrial buildings is vastly dominated by lighting. Also, it is anticipated that the energy demand for cooling will increase and for heating will decrease in the following decades due to climate change. All these features make northlight construction an appealing option.

9.3 Overall conclusions

9.3.1 Single storey industrial buildings

The study demonstrated the benefit associated with exploiting the increased insulation in sandwich panel systems in order to reduce the structure and deliver single storey industrial buildings with greater structural efficiency and reduced embodied carbon. The study found that the greatest potential benefit arises from the use of long span roof envelopes in trussed-roof frames with northlights. The study demonstrated significant benefits associated with this opportunity:

- Up to 60% steelwork saving by mass can be achieved in comparison to traditional portal frame construction. With almost 60% of the constructional steelwork in the UK being used in single storey industrial buildings, the practical impact of such savings is likely to be significant.
- Currently available typical roof sandwich panels can be used as long span systems and suit the optimum span distances for small buildings without modifications. For medium and large buildings, roof sandwich panels would require minimal improvements to achieve the optimum span distances. The study demonstrated that embodied-carbon optimal sandwich panels can be

engineered merely through modest increases in the insulation depth and height of the outer profile.

- Up to 5% savings in embodied carbon when compared to traditional portal frame construction with sandwich panel roof cladding and up to 11.5% savings in embodied carbon when compared to traditional portal frame construction with built-up roof cladding was demonstrated. Savings wer greater for the medium and large building sizes.
- Up to 27% saving in terms of comparative construction cost together with benefits in terms of faster installation were demonstrated. Savings were greater for the medium and large building sizes.
- Significant benefits may exist in terms of operational carbon, particularly for buildings which require cooling and also in terms of lighting and reduction of overheating.
- No significant step-change from the current practice is required for either the design of sandwich panels or frames. This may be appealing for both manufacturers and designers.
- The applicability of the structural scheme is very wide and it may practically be implemented for any type of building size or geometry.

It is highlighted that the energy analyses depend on the grid energy efficiency and source mix assumptions. The assumed energy used in manufacture and the consequent embodied carbon emissions of materials and products could change substantially in the future as the grid energy efficiency and source mix change. Thus, a few percentage points of savings in embodied carbon are subject to change.

Overall, it is recommended that the proposed scheme is adopted if a reduction of embodied carbon and construction costs are pursued for the structural frame and building envelope in single storey industrial buildings.

9.3.2 Holistic structural and energy studies

The study highlights the importance of considering structural design holistically in terms of material interdependence and exploitation for both structural and energy input purposes. Whilst the increase of insulation in the envelope for energy conservation yields increased embodied carbon in the envelope itself, it also presents the opportunity of exploiting its structural capability and redesigning frames with reduced structure, embodied carbon and cost.

For the case study of single storey industrial buildings it was found that considerable savings in embodied carbon and cost are possible when the structural capability of the envelope is exploited and the building frame re-engineered compared against traditional construction solutions. The study demonstrated the feasibility of this approach in the UK context. However, the concept and practice can be extended to other forms of construction and are applicable internationally.

The current research builds on previous studies (Resalati, 2015), which demonstrated the importance of combining operational and embodied carbon analyses in assessing the effectiveness of carbon reduction strategies, as opposed to the conventional 'operational carbon only' methods. The present study shows that a combined building envelope and structure analysis, where the structural capability of the envelope is exploited, creates the opportunity to develop solutions of reduced embodied carbon and cost. When considered together with, and supplementary to, the earlier research on combined operational and embodied carbon for the envelope, the study demonstrates that combined structural and energy analysis for the structure and envelope together are essential to identify true lowest carbon solutions. This represents an essential change of paradigm in terms of carbon and cost and optimisation which needs to be progressed in future work.

9.4 Recommendations for further research

A series of recommendations for further research are proposed, as derived from the context and output of the present study.

9.4.1 Development of low embodied carbon ground floor slabs

The systems review showed that the ground floor slab is the greatest contributor of embodied carbon. Its impact is almost of the same magnitude as that of the whole superstructure and foundations added together due to the very high volume of concrete. Therefore, development of low carbon materials and construction methods for ground floor slabs could facilitate a meaningful and significant embodied carbon reduction of the whole building.

9.4.2 Development of roof sandwich panels with low embodied carbon structural insulation

The embodied carbon review showed that the structural PIR insulation used in the sandwich panels has a similar magnitude of impact to the steel and concrete, particularly for small and medium building sizes. This is due to the very high carbon emissions rate of PIR and the considerable volume of envelope insulation. Alternative structural insulation materials which would combine good structural properties, low thermal transmittance and low density with low emissions rate, such polysterene, could be used to develop sandwich panels with lower embodied carbon impact. Fire requirements would also need to be taken into account, considering that those vary among different countries.

9.4.3 Combined operational and embodied carbon reviews for northlights construction

The study showed that the truss-roofed frames with northlights can yield important embodied carbon and construction cost savings. Their operational energy requirements, however, are fundamentally different compared to the traditional construction with rooflights. Parameters such as truss and glazing height, roof slope and glazing material can influence the building volume, the areas and lengths of thermal bridges, the lighting and daylight factors and the output of PVs, as well as requiring modifications to the structural design and embodied carbon. Consequently, operational energy demand for heating, cooling and lighting can vary significantly, causing net benefit or disbenefit with embodied carbon. A combined analysis of the whole-building's embodied carbon and operational carbon with the aid of dynamic thermal modelling would, therefore, be required. Such analysis would demand function of parameters such as truss depth, frame spacing and choice of northlights glazing materials for various in-use scenarios and environmental conditions, in order to determine the net savings or costs from holistic approach.

9.4.4 Construction economics

The comparative construction cost review was based on assigning cost rates to the material quantities and also quantifying the variation of part counts required on site for the various structural options to provide an indication on speed of construction. A full construction economics study considering cost aspects related to transportation and installation of the different options on site, assuming all variables are known, would provide a more accurate estimate of the comparative construction costs.

9.4.5 System boundaries

A 'cradle-to-gate' approach was adopted in the present research, given that transportation, construction and end-of-life aspects are either project-specific or there are uncertainties within current methodologies. Assuming that all the variables are known, further research incorporating a 'cradle-to-grave' approach in consultation with the steel, concrete and building envelope industries would facilitate a more accurate quantification of the embodied carbon impacts of the various options.

9.4.6 Farm sheds with diaphragm action

Farm sheds in the UK are typically constructed on the basis of closely-spaced portal frames with high-pitch roofs (12.5^o to 17.5^o), comprising timber purlins and cement-board roof cladding. Furthermore, they are typically designed for less onerous load conditions, in specific: lower imposed load as well as snow load due to roof pitch, making wind load more dominant in design. This shifts their governing design modes to typically include combined 'sway' and 'spread' modes (unlike typical industrial buildings which are dominated by 'spread' modes). Furthermore, farm sheds typically have high ratio of frame height / width. All these attributes are particularly beneficial for diaphragm action exploitation and, therefore, present further opportunities, particularly with the aid of sandwich panels which were found ideal means in terms of stiffness.

Furthermore, the use of sandwich panels, may have significant benefits for the control of internal environmental conditions, particularly in moist environments where condensation is often an issue. Further research is required to quantify the potential benefits and identify the barriers.

9.4.7 Frameless buildings

Despite the limited range of applications in terms of building size, frameless buildings with sandwich panels may be a good solution for small and temporary structures, where fast installation is required, such as exhibition kiosks or emergency relief buildings. It was shown that the available sandwich panel technology is adequate without requiring further modifications and that further research should focus on the engineering of connections, more elaborate analytical modelling and, ultimately, prototype building testing. Further research on these aspects would allow for a broader implementation of frameless buildings together with projected savings in terms of structural frame embodied carbon and associated construction costs.

9.4.8 Further diaphragm action opportunities

The diaphragm action study showed that there is limited scope for reduction of structure for the building sizes defined and the code provision approach adopted in the study. There were limitations associated with the use of design standards, which limit the scope only within similar frames in the building, with the chosen typical geometries which do not favour the exploitation of the envelope's in-plane strength and stiffness, and, finally, with the design method adopted in the study (elastic design) which limit the extent of diaphragm action contribution.

A more general parametric study would be required in order to determine the areas and range of meaningful diaphragm action applicability. A broader parametric study, would require using finite element methods to overpass limitations of design standards and realistically assesses the diaphragm action effects on different buildings.

There is potentially greater benefit for structures of different geometries (such as for small length/width and width/height ratios) in which shear girder effects, sway modes and deflection controls (i.e. areas in which diaphragm action is more significant) are more dominant in the design. Furthermore, diaphragm action may have a higher benefit in plastically designed portal frames due to their greater flexibility (due to smaller member sizes) and susceptibility to defections, compared to those elastically designed.

9.4.9 Diaphragm action opportunities for seismic applications

Diaphragm action has greater effect on frames governed by 'sway' modes. There may be particular benefit for buildings in seismic zones when those are designed to resist horizontal earthquake loads and movements. Studies on highly optimised portal frames or re-engineered single storey industrial building schemes such those suggested in the present study may, therefore, be examined. The focus may be on both roof and wall systems used as energy dissipative systems, using primarily but not exclusively sandwich panels. Experimental and numerical studies may be carried out and include: cyclic response of cladding, engineering of cladding-frame connections, ductility aspects, coupled frame-cladding behaviour incorporating hysteretic response and capacity design of structure-diaphragm assemblies. Such investigations may be used to define appropriate behavioural factors for seismic load reduction in design, propose design methods and construction arrangements and, ultimately, demonstrate benefit in terms of structure hardening against seismic loads and potential savings for materials.

9.4.10 Multi-objective optimisation of sandwich panels for cost and embodied carbon

A two-objective optimisation problem was addressed in the current study, seeking for optimum roof panel assemblies in terms of strength and embodied carbon. The current study can form the basis of a multi-objective optimisation problem comprising both cost and embodied carbon aspects. Robust data from manufacturers can be included, incorporating cost implications of various parameters, particularly related to the production line, such as implications of depth increase on production speed, cooling and stacking time. The study would most probably require the use of advanced optimisation techniques, such as genetic algorithms or ant colonies optimisation.

9.4.11 Modelling and structural testing of sandwich panels for out-of-plane restraint of beams

The current design guidance in BS 5950-9:1994 was used in the present research to justify the feasibility of sandwich panels to provide lateral out-of-plane restraint to the chord members of trussed-roof frames. The design methods in the standard assumes a uniform distribution of the compression load arising from the maximum bending moment in the truss girders. A more elaborate model can be developed to incorporate a more realistic and non-uniform force distribution compared to the uniform load assumption proposed in design standards. This would necessitate the investigation of the internal force distribution for the panel diaphragm. A new method would be likely to yield less conservative results and a more realistic stress distribution in the sandwich panels. Furthermore, structural testing would be recommended to validate the theoretical models for sandwich panels, while experimental results in the current literature are limited.

9.4.12 Sensitivity studies for energy grid efficiency and source mix

The current study used Bath's ICE database for the embodied carbon assessments. However, the assumed energy used in manufacture and the consequent embodied carbon emissions of materials and products could change substantially in the future as the grid energy efficiency and source mix change. As the electricity grid emissions in the UK, as well as in rest of Europe, reduce with time and as the energy source mix is already in the trajectory of shift towards less energy produced by fossil fuels and more by renewables and other non-fossil fuel sources, it is likely that the embodied carbon of materials included in Bath's ICE database is subject to major changes in the future. It is, therefore, important that sensitivity studies are undertaken to evaluate the resulting embodied carbon for buildings subject to changes in the energy grid efficiency and source mix.

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Appendix A Specifications of reference sandwich panels and fasteners

The specifications of the modern roof and wall sandwich panel systems used as base cases and references are shown in Sections A.1, A.2 and A.3. Typical fastening products are shown in Section A.4.

A.1 Fully profiled system: steel / PIR (roof panel system)

The nominal geometrical properties of the system are shown in Figure A.1 and Table A.1.

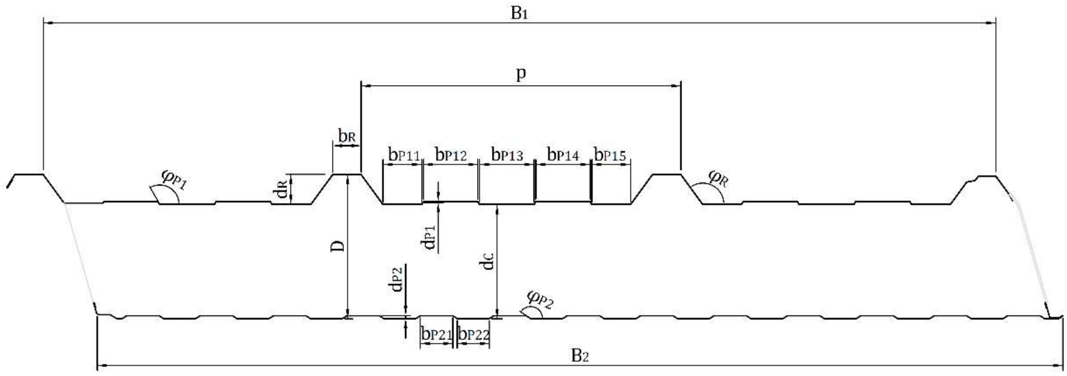

Figure A.1 Cross-section of fully profiled sandwich panel system

Table A.1 Summary of nominal geometrical properties – fully profiled sandwich panel system

Geometrical property	Dimension	Geometrical property	Dimension	Geometrical property	Dimension
B_1	998mm	t_{F1}	0.48mm	t_{F2}	0.34mm
B_2	1012mm	d_R	31.3mm	d_{P2}	3.4mm
p	333mm	b_{R1}	30mm	b_{P21}	34mm
D	90-165mm	φ_R	126°	b_{P22}	34mm
d_C	60 -135mm	d_{P1}	2.8mm	φ_{P2}	147°
		b_{P11}, b_{P15}	40.5mm		
		b_{P12}, b_{P13}, b_{P14}	57mm		
		φ_{P1}	120°		

The nominal mechanical properties of system are summarised in Table A.2.

Table A.2 Summary of nominal mechanical properties – fully profiled sandwich panel system

Material	Mechanical property	Notation	Value
Steel	Elastic modulus	E	210,000N/mm^2
	Yield strength	f_y	220N/mm^2
PIR	Shear modulus	G_v	2.27 N/mm^2
	Shear strength	f_{Cv}	0.100 N/mm^2
	Compressive modulus	E_{Cc}	1.97N/mm^2
	Compressive strength	f_{Cc}	0.11N/mm^2
	Tension modulus	E_{Ct}	1.86N/mm^2
	Density	ρ	40kg/m^3

A.2 Lightly profiled system: steel / PIR (wall panel system)

The nominal geometrical properties of the system are shown in Figure A.2 and Table A.3.

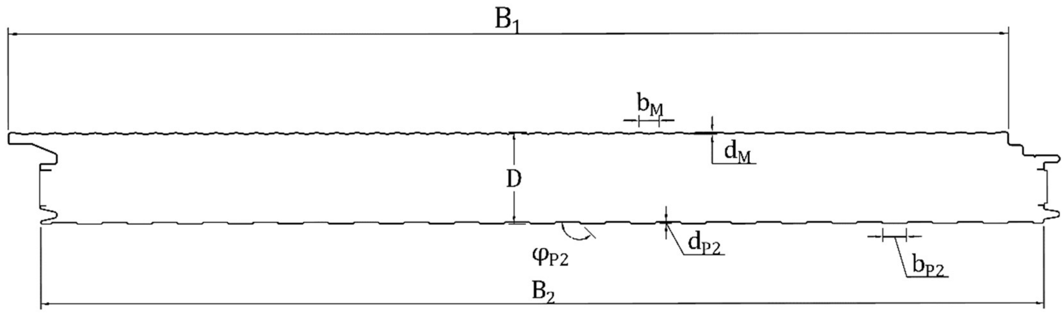

Figure A.2 Cross-section of lightly profiled sandwich panel system

Table A.3 Summary of geometrical properties – lightly profiled sandwich panel system

Geometrical property	Dimension	Geometrical property	Dimension	Geometrical property	Dimension
B_1	996mm	t_{F1}	0.675mm	t_{F2}	0.34mm
B_2	994mm	d_{M1}	1.2mm	d_{P2}	1.5mm
D	70-120mm	b_{M1}	20mm	b_{P21}	23.5mm
				b_{P22}	23.5mm
				φ_{P2}	135^o

The nominal mechanical properties of system are summarised in Table A.4.

Table A.4 Summary of nominal mechanical properties – fully profiled sandwich panel system

Material	Mechanical property	Notation	Value
Steel	Elastic modulus	E	210,000N/mm^2
	Yield strength	f_y	220N/mm^2
PIR	Shear modulus	G_v	2.65N/mm^2
	Shear strength	f_{Cv}	0.102N/mm^2
	Compressive modulus	E_{Cc}	3.76N/mm^2
	Compressive strength	f_{Cc}	0.11N/mm^2
	Tension modulus	E_{Ct}	1.19N/mm^2
	Density	ρ	40kg/m^3

A.3 Flat system: steel / Mineral Wool (wall panel system)

The nominal geometrical properties of the system are shown in Figure A.3 and Table A.5.

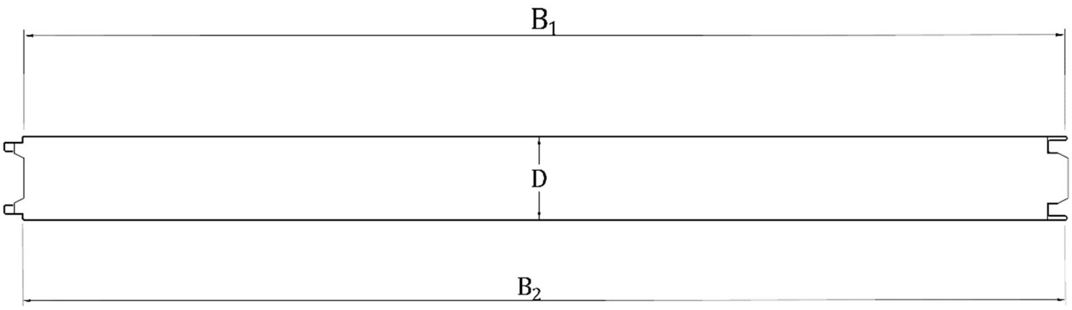

Figure A.3 Cross-section of flat sandwich panel system

Table A.5 Summary of geometrical properties – flat sandwich panel system

Geometrical property	Dimension	Geometrical property	Dimension	Geometrical property	Dimension
B1	1000mm	t_{F1}	0.70mm	t_{F2}	0.45mm
B2	1000mm				
D	75-240mm				

The nominal mechanical properties of system are summarised in Table A.6.

Table A.6 Summary of nominal mechanical properties – flat sandwich panel system

Material	Mechanical property	Notation	Value
Steel	Elastic modulus	E	210,000N/mm^2
	Yield strength	f_y	220N/mm^2
Mineral Wool	Density	ρ	120kg/m^3

A.4 Fasteners

Typical edge and seam fastener specifications used for roof and wall sandwich panels are shown in Table A.7 and Table A.8, extracted from manufacturer's technical literature (SFS, 2002, 2005).

Table A.7 Edge fastener specification – extracted from SFS (2002)

Description	Stainless steel through fastener with washer		
Diameter	5.5mm		
Length	To suit panel thickness		
Pull-out load	*Base thickness (steel)*	*Mean*	*Standard deviation*
	1.5mm	2,300N	85
	2.0mm	4,000N	150
	3.0mm	7,200N	185
	4.0mm	10,280N	1035
Pull-over load (for washer S16)	*Sheet thickness*	*Mean*	*Standard deviation*
	0.50mm	4100N	495
	0.62mm	4800N	345
	0.75mm	6300N	540
	0.87mm	6600N	530
Tensile breaking load		*Mean*	*Standard deviation*
		17,500N	Not included
Shear breaking load		*Mean*	*Standard deviation*
		10,900N	Not included

Table A.8 Seam fastener specification – extracted from (SFS, 2005)

Description	Stainless steel self-tapping screw with washer		
Diameter	4.8mm		
Length	22mm		
Pull-out load	*Base thickness (steel)*	*Mean*	*Standard deviation*
	0.40mm	525	22
	2x0.50mm	682	27
	2x0.63mm	1015	71
	2x0.75mm	1171	61
	2x1.00mm	1687	57
Shear load	Sheet / base thickness (steel)	*Mean*	*Standard deviation*
	0.63mm / 0.63mm	1524	64
Tensile breaking load		*Mean*	*Standard deviation*
		9,800N	Not included
Shear breaking load		*Mean*	*Standard deviation*
		5,900N	Not included

A.5 Design example: single-span fully profiled roof sandwich panel in bending

The panel arrangement and loading for the design example is shown in Figure A.4.

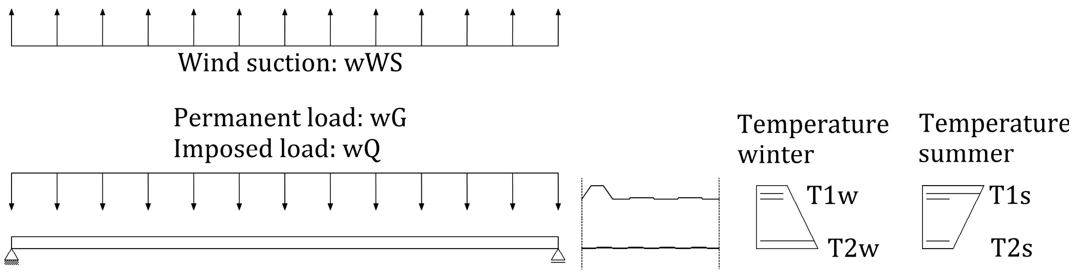

Figure A.4 Design example: panel arrangement and loading

The cross-section of the panel and the notation of geometrical properties used in this design example is shown in Figure A.5.

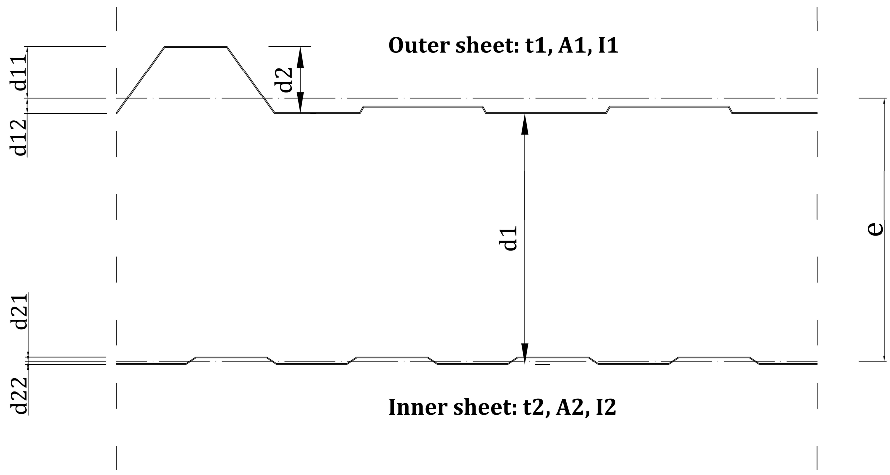

Figure A.5 Design example: panels cross-section and notation of geometrical properties

The mechanical properties, resistances and material safety factors assumed in this example are according to typical nominal manufacturers' values. It is noted that in this particular case the outer sheet is fully effective when under compression, hence yielding occurs prior to buckling. Although this is typical for modern fully profiled sandwich panels, it may not necessarily be true for all panel cases and the assumption should be validated by testing.

The design example is according to BS EN 14509:2013. The process followed is based on Davies *et. al.* (2001).

The calculation example is shown as programmed in a mathematical processor.

A.5.1 Load combinations

The load combinations are according to BS EN 1990. Load factors have also been used BS EN 14509:2013. The full list of load combinations is shown here:

- Permanent + Imposed
- Permanent + Imposed + Temperature (accompanying)
- Permanent + Wind
- Permanent + Wind + Temperature (accompanying)
- Permanent + Snow
- Permanent + Snow + Temperature (accompanying)
- Permanent + Snow (leading) + Wind (accompanying)
- Permanent + Snow (leading) + Wind (accompanying) + Temperature accompanying)
- Permanent + Wind (leading) + Snow (Accompanying)
- Permanent + Wind (leading) + Snow (Accompanying) + Temperature accompanying)
- Permanent + Temperature

For the assumed location, building geometry and use the following actions apply (see Appendix B):

Permanent actions

- Self-weight: w_G= 0.129kN/m^2

Variable actions

- Imposed load: w_Q= 0.6kN/m^2 (for roofs accessible only for maintenance
- Wind pressure: w_{w+}= 0.21kN/m^2
- Wind suction: w_{w-}= 0.41kN/m^2
- Snow load: w_S= 0.4kN/m^2
- Temperature in summer: T_1=55^{o}C, T_2=25^{o}C (for colour group II, medium colours)
- Temperature in winter: T_1=-10^{o}C, T_2=20^{o}C

From inspection of the action magnitudes, the imposed load is the governing variable action for pressure. Wind suction is the only variable action for uplift. The effects of the temperature need to be taken into account for both cases.

Only the dominant load combinations are shown in this example, these are:

- Permanent + Imposed
- Permanent + Imposed + Temperature
- Permanent + Wind suction
- Permanent + Wind suction + Temperature

No further calculations are shown in this design example for the snow and wind pressure actions.

A.5.2 Calculation

The design example is shown herein as extracted from the mathematical processor.

Numerical example: Single-span fully profiled roof sandwich panel

Panel properties

Panel arragnement

"Span" $Lo := 6.67\ m$ $L := 1 \cdot Lo$ "For 1-span"

"Support width" $Lse := 100\ mm$

"Core depth" $d1 := 135\ mm$

Panel cross-section

Outer sheet

"Sheet thickness" $t1 := 0.48\ mm$

"Sheet area" $A1 := 511\ mm^2$

"Moment of inertia" $I1 := 51775\ mm^4$

"Profile depth" $d2 := 31.3\ mm$

"Distance to centroid of profile" $d12 := 8.5\ mm$

$d11 := d2 - d12 = 22.8\ mm$

Inner sheet

"Sheet thickness" $t2 := 0.34\ mm$

"Sheet area" $A2 := 309.74\ mm^2$

"Moment of inertia" $I2 := 588.3\ mm^4$

"DIstance between centroids of sheets" $e := d1 + d12 + 0.5 \cdot (t1 + t2)$ $e = 0.144\ m$

"Core area" $Ac := 1\ m \cdot e$ $Ac = \left(1.439 \cdot 10^5\right)\ mm^2$

Mechanical properties (design values)

Core

"Shear modulus" $G := 2.27 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$

"Shear strength" $\tau C := 0.100 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$

"Compression strength" $fCc := 0.110 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$

"Distributional parameter for PIR" $k := 0.5$

Sheets

"Modulus of elasticity" $E := 210000 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$

"Yield strength - outer sheet" $fyF1 := 220 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$

"Yield strength - inner sheet" $fyF2 := 220 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$

"Compresion strength - outer sheet" $\sigma wF1 := 220 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$ "Note: outer sheet fully effective"

"Compression strength - inner sheet" $\sigma wF2 := 183 \dfrac{\mathbf{N}}{\mathbf{mm}^2}$

"Coefficient of thermal expansion - outer sheet" $\alpha F1 := \dfrac{12 \cdot 10^{-6}}{1\ \mathbf{°C}}$

"Coefficient of thermal expansion - inner sheet" $\alpha F2 := \dfrac{12 \cdot 10^{-6}}{1\ \mathbf{°C}}$

Fasteners

"Fastener strength in wind uplift" $fFRfast := 2.38\ \mathbf{kN}$

"Number of fasteners at support" $nfast := 3$

Initial desing parameters

"Bending stiffness - flange part" $BD := E \cdot I1 + E \cdot I2$ $BD = (1.1 \cdot 10^{10})\ N \cdot mm^2$

"Bending stiffness - sandwich part" $BS := \dfrac{E \cdot A1 \cdot E \cdot A2}{E \cdot A1 + E \cdot A2} \cdot e^2$ $BS = (8.387 \cdot 10^{11})\ N \cdot mm^2$

"Bending stiffness - total" $B := BD + BS$ $B = (8.497 \cdot 10^{11})\ N \cdot mm^2$

"Effective shear modulus" $Geff := G \cdot \dfrac{e}{d1}$ $Geff = 2.42\ \dfrac{N}{mm^2}$

$\alpha := \dfrac{BD}{BS}$ $\alpha = 0.013$

$\beta o := \dfrac{BS}{(Lo)^2 \cdot Geff \cdot Ac}$ $\beta o = 0.054$

$\beta := 1 \cdot \beta o$ $\beta = 0.054$ "For 1-span"

$\lambda o := \sqrt{\dfrac{1+\alpha}{\alpha \cdot \beta o}}$ $\lambda o = 37.781$

$\lambda := 1 \cdot \lambda o$ $\lambda = 37.781$ "For 1-span"

Actions

"Self weight" $wG := 0.129\ \dfrac{kN}{m}$

"Imposed load" $wQ := 0.60\ \dfrac{kN}{m}$

"Wind suction" $wWS := -0.55\ \dfrac{kN}{m}$

"Summer temperature" "Outer sheet" $T1s := 65\ °C$ "Colour group II: medium"

"Inner sheet" $T2s := 25\ °C$

"Winter temperature" "Outer sheet" $T1w := -10\ °C$

"Inner sheet" $T2w := 20\ °C$

Load factors

"Permanent load" $\gamma G := 1.35$ "Unfavourable; 1.0 for favourable"

"Imposed / wind load" $\gamma Q := 1.50$

"Temperature load" $\gamma T := 1.50$

"Accompanying variable action" $\psi := 0.6$

Material safety factors

"Yielding of steel sheet" $\gamma MFt := 1.10$

"Wrinkling / compression of steel sheet in span" $\gamma MFc := 1.25$

"Shear / crushing of core" $\gamma MC := 1.25$

"Fastener" $\gamma MFast := 1.33$

Design resistances and limits

Ultimate Limit States (ULS)

"Sheet in compression" $RdF1compr := \dfrac{\sigma wF1}{\gamma MFc} = 176\ \mathbf{MPa}$

$RdF2compr := \dfrac{\sigma wF2}{\gamma MFc} = 146.4\ \mathbf{MPa}$

"Sheet in tension" $RdF1tension := \dfrac{fyF1}{\gamma MFt} = 200\ \mathbf{MPa}$

$RdF2tension := \dfrac{fyF2}{\gamma MFt} = 200\ \mathbf{MPa}$

"Core shear" $Rd\tau C := \dfrac{\tau C}{\gamma MC} = 0.08\ \mathbf{MPa}$

"Core crushing" $RdfCc := \frac{fCc}{\gamma MC} = 0.088\ \mathbf{MPa}$

"Fasteners in uplift" $RdFRfast := \frac{nfast \cdot fFRfast}{\gamma MFast} = 5.368\ \mathbf{kN}$

Serviceability Limit States (SLS)

"Deflection limit in pressure" $\Delta pressureLimit := \frac{L}{200} = 33.35\ \mathbf{mm}$

"Deflection limit in uplift" $\Delta upliftLimit := \frac{L}{150} = 44.467\ \mathbf{mm}$

Stresses, forces and deflections for individual actions (unfactored)

Action: Self weight

"Unfactored bending moments"

$$MDwG(\xi) := wG \cdot L^2 \cdot \frac{\alpha}{1+\alpha} \cdot \left(0.5 \cdot \xi \cdot (1-\xi) + \frac{\cosh(0.5 \cdot \lambda) - \cosh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\alpha \cdot \lambda^2 \cdot \cosh(0.5 \cdot \lambda)}\right)$$

$$MSwG(\xi) := wG \cdot L^2 \cdot \frac{1}{1+\alpha} \cdot \left(0.5 \cdot \xi \cdot (1-\xi) - \frac{\cosh(0.5 \cdot \lambda) - \cosh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\lambda^2 \cdot \cosh(0.5 \cdot \lambda)}\right)$$

"At mid-span" $MDwG\left(\frac{1}{2}\right) = 0.013\ \boldsymbol{kN \cdot m}$ "in flange part"

$MSwG\left(\frac{1}{2}\right) = 0.704\ \boldsymbol{kN \cdot m}$ "in sandwich part"

"Unfactored bending stresses"

$$\sigma F11wG(\xi) := -MDwG(\xi) \cdot \frac{d11}{I1} - MSwG(\xi) \cdot \frac{1}{A1 \cdot e}$$ "Outer sheet - at crest"

$$\sigma F12wG(\xi) := MDwG(\xi) \cdot \frac{d12}{I1} - MSwG(\xi) \cdot \frac{1}{A1 \cdot e}$$ "Outer sheet - at trough"

$$\sigma F2wG(\xi) := MSwG(\xi) \cdot \frac{1}{A2 \cdot e}$$

"At mid-span"

"Outer sheet" $\sigma F11wG\left(\frac{1}{2}\right) = -15.411\ \boldsymbol{MPa}$ "at crest"

$\sigma F12wG\left(\frac{1}{2}\right) = -7.399\ \boldsymbol{MPa}$ "at trough"

$\sigma F1wG := min\left(\sigma F11wG\left(\frac{1}{2}\right), \sigma F12wG\left(\frac{1}{2}\right)\right) = -15.411\ \boldsymbol{MPa}$ "max stress"

"Inner sheet" $\sigma F2wG\left(\frac{1}{2}\right) = 15.797\ \boldsymbol{MPa}$

"Unfactored core shear force"

$$VSwG(\xi) := wG \cdot L \cdot \frac{1}{1+\alpha} \cdot \left(0.5 \cdot (1-2\ \xi) + \frac{\sinh(0.5 \cdot \lambda \cdot (1-2 \cdot \xi))}{\lambda \cdot \cosh(0.5 \cdot \lambda)}\right)$$

"At support" $VSwG(0) = 0.447\ \mathbf{kN}$ "in core - sandwich part"

"Unfactored core shear stresses"

$$\tau CwG(\xi) := \frac{VSwG(\xi)}{Ac}$$

"At support" $\tau CwG(0) = 0.003\ \mathbf{MPa}$ "in core"

"Unfactored support reaction"

$$FRwG(\xi) := \frac{wG \cdot L}{2}$$

"At support" $FRwG(0) = 0.43\ \mathbf{kN}$

"Unfactored core crushing"

$$fCcwG(\xi) := \frac{FRwG(\xi)}{1\ \mathbf{m} \cdot (Lse + 0.5 \cdot k \cdot min(100\ \mathbf{mm}, e))}$$

"At support" $fCcwG(0) = 0.003\ \mathbf{MPa}$

"Unfactored deflections"

$$\Delta wG(\xi) := wG \cdot \frac{L^4}{B} \cdot \left(\frac{1}{24} \cdot \xi \cdot (1 - 2 \cdot \xi^2 + \xi^3) + \xi \cdot \frac{(1-\xi)}{2 \cdot \alpha \cdot \lambda^2} - \frac{\left(\cosh\left(\frac{\lambda}{2}\right) - \cosh\left(\lambda \cdot \frac{(1-2 \cdot \xi)}{2}\right)\right)}{\alpha \cdot \lambda^4\ \cosh\left(\frac{\lambda}{2}\right)}\right)$$

"At mid-span" $\Delta wG\left(\frac{1}{2}\right) = 5.908\ \mathbf{mm}$

Load case: imposed load

"Unfactored bending moments"

$$MDwQ(\xi) := wQ \cdot L^2 \cdot \frac{\alpha}{1+\alpha} \cdot \left(0.5 \cdot \xi \cdot (1-\xi) + \frac{\cosh(0.5 \cdot \lambda) - \cosh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\alpha \cdot \lambda^2 \cdot \cosh(0.5 \cdot \lambda)}\right)$$

$$MSwQ(\xi) := wQ \cdot L^2 \cdot \frac{1}{1+\alpha} \cdot \left(0.5 \cdot \xi \cdot (1-\xi) - \frac{\cosh(0.5 \cdot \lambda) - \cosh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\lambda^2 \cdot \cosh(0.5 \cdot \lambda)}\right)$$

"At mid-span" $MDwQ\left(\frac{1}{2}\right) = 0.062\ \boldsymbol{kN \cdot m}$ "in flange part"

$MSwQ\left(\frac{1}{2}\right) = 3.275\ \boldsymbol{kN \cdot m}$ "in sandwich part"

"Unfactored bending stresses"

"Outer sheet"

$$\sigma F11wQ(\xi) := -MDwQ(\xi) \cdot \frac{d11}{I1} - MSwQ(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at crest"

$$\sigma F12wQ(\xi) := MDwQ(\xi) \cdot \frac{d12}{I1} - MSwQ(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at trough"

"Inner sheet"

$$\sigma F2wQ(\xi) := MSwQ(\xi) \cdot \frac{1}{A2 \cdot e}$$

"At mid-span"

"Outer sheet" $\sigma F11wQ\left(\frac{1}{2}\right) = -71.679\ \boldsymbol{MPa}$ "at crest"

$\sigma F12wQ\left(\frac{1}{2}\right) = -34.416\ \boldsymbol{MPa}$ "at trough"

$\sigma F1wQ := min\left(\sigma F11wQ\left(\frac{1}{2}\right), \sigma F12wQ\left(\frac{1}{2}\right)\right) = -71.679\ \boldsymbol{MPa}$ "total"

"Inner sheet" $\sigma F2wQ\left(\frac{1}{2}\right) = 73.473\ \boldsymbol{MPa}$

"Unfactored core shear force"

$$VSwQ(\xi) := wQ \cdot L \cdot \frac{1}{1+\alpha} \cdot \left(0.5 \cdot (1 - 2\ \xi) + \frac{\sinh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\lambda \cdot \cosh(0.5 \cdot \lambda)}\right)$$

"At support" $VSwQ(0) = 2.08\ \boldsymbol{kN}$ "in core - sandwich part"

"Unfactored core shear stresses"

$$\tau CwQ(\xi) := \frac{VSwQ(\xi)}{Ac}$$

"At support" $\tau CwQ(0) = 0.014\ \boldsymbol{MPa}$ "in core"

"Unfactored support reaction"

$$FRwQ(\xi) := \frac{wQ \cdot L}{2}$$

"At support" $FRwQ(0) = 2.001\ \boldsymbol{kN}$

"Unfactored core crushing"

$$fCcwQ(\xi) := \frac{FRwQ(\xi)}{1\ \boldsymbol{m} \cdot (Lse + 0.5 \cdot k \cdot min(100\ \boldsymbol{mm}, e))}$$

"At support" $fCcwQ(0) = 0.016\ \boldsymbol{MPa}$

"Unfactored deflections"

$$\Delta wQ(\xi) := wQ \cdot \frac{L^4}{B} \cdot \left(\frac{1}{24} \cdot \xi \cdot \left(1 - 2 \cdot \xi^2 + \xi^3\right) + \xi \cdot \frac{(1-\xi)}{2 \cdot \alpha \cdot \lambda^2} - \frac{\left(\cosh\left(\frac{\lambda}{2}\right) - \cosh\left(\lambda \cdot \frac{(1 - 2 \cdot \xi)}{2}\right)\right)}{\alpha \cdot \lambda^4\ \cosh\left(\frac{\lambda}{2}\right)}\right)$$

"At mid-span" $\Delta wQ\left(\frac{1}{2}\right) = 27.481\ \boldsymbol{mm}$

Action: wind suction

"Unfactored bending moments"

$$MDwWS(\xi) := wWS \cdot L^2 \cdot \frac{\alpha}{1+\alpha} \cdot \left(0.5 \cdot \xi \cdot (1-\xi) + \frac{\cosh(0.5 \cdot \lambda) - \cosh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\alpha \cdot \lambda^2 \cdot \cosh(0.5 \cdot \lambda)}\right)$$

$$MSwWS(\xi) := wWS \cdot L^2 \cdot \frac{1}{1+\alpha} \cdot \left(0.5 \cdot \xi \cdot (1-\xi) - \frac{\cosh(0.5 \cdot \lambda) - \cosh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\lambda^2 \cdot \cosh(0.5 \cdot \lambda)}\right)$$

"At mid-span" $MDwWS\left(\frac{1}{2}\right) = -0.057\ \mathbf{kN \cdot m}$ "in flange part"

$MSwWS\left(\frac{1}{2}\right) = -3.002\ \mathbf{kN \cdot m}$ "in sandwich part"

"Unfactored bending stresses"

"Outer sheet"

$$\sigma F11wWS(\xi) := -MDwWS(\xi) \cdot \frac{d11}{I1} - MSwWS(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at crest"

$$\sigma F12wWS(\xi) := MDwWS(\xi) \cdot \frac{d12}{I1} - MSwWS(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at trough"

"Inner sheet"

$$\sigma F2wWS(\xi) := MSwWS(\xi) \cdot \frac{1}{A2 \cdot e}$$

"At mid-span"

"Top sheet" $\sigma F11wWS\left(\frac{1}{2}\right) = 65.706\ \mathbf{MPa}$ "at crest"

$\sigma F12wWS\left(\frac{1}{2}\right) = 31.548\ \mathbf{MPa}$ "at trough"

$\sigma F1wWS := \max\left(\sigma F11wWS\left(\frac{1}{2}\right), \sigma F12wWS\left(\frac{1}{2}\right)\right) = 65.706\ \mathbf{MPa}$ "Max stress"

"Bottom sheet" $\sigma F2wWS\left(\frac{1}{2}\right) = -67.35\ \mathbf{MPa}$

"Unfactored core shear force"

$$VSwWS(\xi) := wWS \cdot L \cdot \frac{1}{1+\alpha} \cdot \left(0.5 \cdot (1-2\ \xi) + \frac{\sinh(0.5 \cdot \lambda \cdot (1-2 \cdot \xi))}{\lambda \cdot \cosh(0.5 \cdot \lambda)}\right)$$

"At support" $VSwWS(0) = -1.906\ \boldsymbol{kN}$ "in core - sandwich part"

"Unfactored core shear stresses"

$$\tau CwWS(\xi) := \frac{VSwQ(\xi)}{Ac}$$

"At support" $\tau CwWS(0) = 0.014\ \boldsymbol{MPa}$ "in core"

"Unfactored support reaction"

$$FRwWS(\xi) := \frac{wWS \cdot L}{2}$$

"At support" $FRwWS(0) = -1.834\ \boldsymbol{kN}$

"Unfactored deflections"

$$\Delta wWS(\xi) := wWS \cdot \frac{L^4}{B} \cdot \left(\frac{1}{24} \cdot \xi \cdot \left(1 - 2 \cdot \xi^2 + \xi^3\right) + \xi \cdot \frac{(1-\xi)}{2 \cdot \alpha \cdot \lambda^2} - \frac{\left(\cosh\left(\frac{\lambda}{2}\right) - \cosh\left(\lambda \cdot \frac{(1-2 \cdot \xi)}{2}\right)\right)}{\alpha \cdot \lambda^4\ \cosh\left(\frac{\lambda}{2}\right)}\right)$$

"At mid-span" $\Delta wWS\left(\frac{1}{2}\right) = -25.191\ \boldsymbol{mm}$

Action: temperature summer

"Curvature due to temperature"

$$\theta s := \frac{\alpha F2 \cdot T2s - \alpha F1 \cdot T1s}{e} = -1.217 \cdot 10^{-5} \ \frac{1}{\mathbf{m}}$$

"Unfactored bending moments"

$$MDTs(\xi) := \theta s \cdot \frac{\alpha \cdot BS}{1+\alpha} \cdot \left(\frac{\cosh(0.5 \cdot \lambda) - \cosh(\lambda \cdot (1 - 2 \cdot \xi))}{\cosh(0.5 \cdot \lambda)} \right)$$

$$MSTs(\xi) := -\theta s \cdot \frac{\alpha \cdot BS}{1+\alpha} \cdot \left(\frac{\cosh(0.5 \cdot \lambda) - \cosh(\lambda \cdot (1 - 2 \cdot \xi))}{\cosh(0.5 \cdot \lambda)} \right)$$

"At mid-span" $\quad MDTs\left(\frac{1}{2}\right) = -1.321 \cdot 10^{-4} \ \mathbf{kN \cdot m}$ "in flange part"

$MSTs\left(\frac{1}{2}\right) = \left(1.321 \cdot 10^{-4}\right) \ \mathbf{kN \cdot m}$ "in sandwich part"

"Unfactored bending stresses"

"Outer sheet"

$$\sigma F11Ts(\xi) := -MDTs(\xi) \cdot \frac{d11}{I1} - MSTs(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at crest"

$$\sigma F12Ts(\xi) := MDTs(\xi) \cdot \frac{d12}{I1} - MSTs(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at trough"

"Inner sheet"

$$\sigma F2Ts(\xi) := MSTs(\xi) \cdot \frac{1}{A2 \cdot e}$$

"At mid-span"

"Outer sheet" $\sigma F11Ts\left(\frac{1}{2}\right) = 0.056\ \mathbf{MPa}$ "at crest"

$\sigma F12Ts\left(\frac{1}{2}\right) = -0.023\ \mathbf{MPa}$ "at trough"

$\sigma F1Ts := \max\left(\sigma F11Ts\left(\frac{1}{2}\right), \sigma F12Ts\left(\frac{1}{2}\right)\right) = 0.056\ \mathbf{MPa}$ "Max stress"

"Inner sheet" $\sigma F2Ts\left(\frac{1}{2}\right) = 0.003\ \mathbf{MPa}$

"Unfactored core shear force"

$$VSTs(\xi) := \theta s \cdot BS \cdot \frac{1}{\beta \cdot \lambda \cdot L} \cdot \left(\frac{\sinh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\cosh(0.5 \cdot \lambda)}\right)$$

"At support" $VSTs(0) = -7.48 \cdot 10^{-4}\ \mathbf{kN}$ "in core - sandwich part"

"Unfactored core shear stresses"

$$\tau CTs(\xi) := \frac{VSTs(\xi)}{Ac}$$

"At support" $\tau CTs(0) = -5.198 \cdot 10^{-6}\ \mathbf{MPa}$ "in core"

"Unfactored deflections"

$$\Delta Ts(\xi) := \frac{\theta s \cdot L^2}{1 + \alpha} \cdot \left(\left(0.5 \cdot \xi \cdot (1 - \xi) - \frac{1}{\lambda^2} \cdot \frac{\cosh(0.5 \cdot \lambda) - \cosh(0.5 \cdot \lambda \cdot (1 - 2 \cdot \xi))}{\cosh(0.5 \cdot \lambda)}\right)\right)$$

"At mid-span" $\Delta Ts\left(\frac{1}{2}\right) = -0.066\ \mathbf{mm}$

Action: temperature winter

"Curvature due to temperature"

$$\theta w := \frac{\alpha F2 \cdot T2w - \alpha F1 \cdot T1w}{e} = \left(9.125 \cdot 10^{-6}\right) \frac{1}{m}$$

"Unfactored bending moments"

$$MDTw(\xi) := \theta w \cdot \frac{\alpha \cdot BS}{1+\alpha} \cdot \left(\frac{\cosh(0.5 \cdot \lambda) - \cosh(\lambda \cdot (1 - 2 \cdot \xi))}{\cosh(0.5 \cdot \lambda)}\right)$$

$$MSTw(\xi) := -\theta w \cdot \frac{\alpha \cdot BS}{1+\alpha} \cdot \left(\frac{\cosh(0.5 \cdot \lambda) - \cosh(\lambda \cdot (1 - 2 \cdot \xi))}{\cosh(0.5 \cdot \lambda)}\right)$$

"At mid-span" $MDTw\left(\frac{1}{2}\right) = \left(9.904 \cdot 10^{-5}\right) \ kN \cdot m$ "in flange part"

$MSTw\left(\frac{1}{2}\right) = -9.904 \cdot 10^{-5} \ kN \cdot m$ "in sandwich part"

"Unfactored bending stresses"

"Outer sheet"

$$\sigma F11Tw(\xi) := -MDTw(\xi) \cdot \frac{d11}{I1} - MSTw(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at crest"

$$\sigma F12Tw(\xi) := MDTw(\xi) \cdot \frac{d12}{I1} - MSTw(\xi) \cdot \frac{1}{A1 \cdot e}$$ "at trough"

"Inner sheet"

$$\sigma F2Tw(\xi) := MSTw(\xi) \cdot \frac{1}{A2 \cdot e}$$

"At mid-span"

"Outer sheet" $\sigma F11Tw\left(\frac{1}{2}\right)=-0.042\ \boldsymbol{MPa}$ "at crest"

$\sigma F12Tw\left(\frac{1}{2}\right)=0.018\ \boldsymbol{MPa}$ "at trough"

$\sigma F1Tw:=min\left(\sigma F11Tw\left(\frac{1}{2}\right),\sigma F12Tw\left(\frac{1}{2}\right)\right)=-0.042\ \boldsymbol{MPa}$ "Max stress"

"Inner sheet" $\sigma F2Tw\left(\frac{1}{2}\right)=-0.002\ \boldsymbol{MPa}$

"Unfactored shear force"

$$VSTw(\xi):=\theta w\cdot BS\cdot\frac{1}{\beta\cdot\lambda\cdot L}\cdot\left(\frac{\sinh(0.5\cdot\lambda\cdot(1-2\cdot\xi))}{\cosh(0.5\cdot\lambda)}\right)$$

"At support" $VSTw(0)=\left(5.61\cdot10^{-4}\right)\ \boldsymbol{kN}$ "in core - sandwich part"

"Unfactored shear stresses"

$$\tau CTw(\xi):=\frac{VSTw(\xi)}{Ac}$$

"At support" $\tau CTw(0)=\left(3.898\cdot10^{-6}\right)\ \boldsymbol{MPa}$ "in core"

"Deflections"

$$\Delta Tw(\xi):=\frac{\theta w\cdot L^2}{1+\alpha}\cdot\left(\left(0.5\cdot\xi\cdot(1-\xi)-\frac{1}{\lambda^2}\cdot\frac{\cosh(0.5\cdot\lambda)-\cosh(0.5\cdot\lambda\cdot(1-2\cdot\xi))}{\cosh(0.5\cdot\lambda)}\right)\right)$$

"At mid-span" $\Delta Tw\left(\frac{1}{2}\right)=0.05\ \boldsymbol{mm}$

Load combinations

Outer sheet stress

$$LC1\Sigma F1 := \gamma G \cdot \sigma F1wG + \gamma Q \cdot \sigma F1wQ + \psi \cdot \gamma T \cdot \sigma F1Tw = -128.362\ \mathbf{MPa}$$

$$LC2\Sigma F1 := \gamma G \cdot \sigma F1wG + \psi \cdot \gamma Q \cdot \sigma F1wQ + \gamma T \cdot \sigma F1Tw = -85.38\ \mathbf{MPa}$$

$$LC3\Sigma F1 := \gamma G \cdot \sigma F1wG + \gamma Q \cdot \sigma F1wQ + \psi \cdot \gamma T \cdot \sigma F1Ts = -128.273\ \mathbf{MPa}$$

$$LC4\Sigma F1 := \gamma G \cdot \sigma F1wG + \psi \cdot \gamma Q \cdot \sigma F1wQ + \gamma T \cdot \sigma F1Ts = -85.232\ \mathbf{MPa}$$

$$LC5\Sigma F1 := \gamma G \cdot \sigma F1wG + \gamma Q \cdot \sigma F1wQ = -128.324\ \mathbf{MPa}$$

$$LC6\Sigma F1 := 1 \cdot \sigma F1wG + \gamma Q \cdot \sigma F1wWS = 83.148\ \mathbf{MPa}$$

$$LC7\Sigma F1 := 1 \cdot \sigma F1wG + \gamma Q \cdot \sigma F1wWS + \psi \cdot \gamma T \cdot \sigma F1Ts = 83.199\ \mathbf{MPa}$$

$$LC8\Sigma F1 := 1 \cdot \sigma F1wG + \psi \cdot \gamma Q \cdot \sigma F1wWS + \gamma T \cdot \sigma F1Tw = 43.661\ \mathbf{MPa}$$

Inner sheet stress

$$LC1\Sigma F2 := \gamma G \cdot \sigma F2wG\left(\frac{1}{2}\right) + \gamma Q \cdot \sigma F2wQ\left(\frac{1}{2}\right) + \psi \cdot \gamma T \cdot \sigma F2Tw\left(\frac{1}{2}\right) = 131.533\ \mathbf{MPa}$$

$$LC2\Sigma F2 := \gamma G \cdot \sigma F2wG\left(\frac{1}{2}\right) + \psi \cdot \gamma Q \cdot \sigma F2wQ\left(\frac{1}{2}\right) + \gamma T \cdot \sigma F2Tw\left(\frac{1}{2}\right) = 87.448\ \mathbf{MPa}$$

$$LC3\Sigma F2 := \gamma G \cdot \sigma F2wG\left(\frac{1}{2}\right) + \gamma Q \cdot \sigma F2wQ\left(\frac{1}{2}\right) + \psi \cdot \gamma T \cdot \sigma F2Ts\left(\frac{1}{2}\right) = 131.537\ \mathbf{MPa}$$

$$LC4\Sigma F2 := \gamma G \cdot \sigma F2wG\left(\frac{1}{2}\right) + \psi \cdot \gamma Q \cdot \sigma F2wQ\left(\frac{1}{2}\right) + \gamma T \cdot \sigma F2Ts\left(\frac{1}{2}\right) = 87.455\ \mathbf{MPa}$$

$$LC5\Sigma F2 := \gamma G \cdot \sigma F2wG\left(\frac{1}{2}\right) + \gamma Q \cdot \sigma F2wQ\left(\frac{1}{2}\right) = 131.535\ \mathbf{MPa}$$

$$LC6\Sigma F2 := 1 \cdot \sigma F2wG\left(\frac{1}{2}\right) + \gamma Q \cdot \sigma F2wWS\left(\frac{1}{2}\right) = -85.228\ \mathbf{MPa}$$

$$LC7\Sigma F2 := 1 \cdot \sigma F2wG\left(\frac{1}{2}\right) + \gamma Q \cdot \sigma F2wWS\left(\frac{1}{2}\right) + \psi \cdot \gamma T \cdot \sigma F2Ts\left(\frac{1}{2}\right) = -85.226\ \mathbf{MPa}$$

$$LC8\Sigma F2 := 1 \cdot \sigma F2wG\left(\frac{1}{2}\right) + \psi \cdot \gamma Q \cdot \sigma F2wWS\left(\frac{1}{2}\right) + \gamma T \cdot \sigma F2Tw\left(\frac{1}{2}\right) = -44.822\ \mathbf{MPa}$$

Core shear stress

$$LC1\Sigma\tau C := \gamma G \cdot \tau CwG(0) + \gamma Q \cdot \tau CwQ(0) + \psi \cdot \gamma T \cdot \tau CTw(0) = 0.026\ \mathbf{MPa}$$

$$LC2\Sigma\tau C := \gamma G \cdot \tau CwG(0) + \psi \cdot \gamma Q \cdot \tau CwQ(0) + \gamma T \cdot \tau CTw(0) = 0.017\ \mathbf{MPa}$$

$$LC3\Sigma\tau C := \gamma G \cdot \tau CwG(0) + \gamma Q \cdot \tau CwQ(0) + \psi \cdot \gamma T \cdot \tau CTs(0) = 0.026\ \mathbf{MPa}$$

$$LC4\Sigma\tau C := \gamma G \cdot \tau CwG(0) + \psi \cdot \gamma Q \cdot \tau CwQ(0) + \gamma T \cdot \tau CTs(0) = 0.017\ \mathbf{MPa}$$

$$LC5\Sigma\tau C := \gamma G \cdot \tau CwG(0) + \gamma Q \cdot \tau CwQ(0) = 0.026\ \mathbf{MPa}$$

$$LC6\Sigma\tau C := 1 \cdot \tau CwG(0) + \gamma Q \cdot \tau CwWS(0) = 0.025\ \mathbf{MPa}$$

$$LC7\Sigma\tau C := 1 \cdot \tau CwG(0) + \gamma Q \cdot \tau CwWS(0) + \psi \cdot \gamma T \cdot \tau CTs(0) = 0.025\ \mathbf{MPa}$$

$$LC8\Sigma\tau C := 1 \cdot \tau CwG(0) + \psi \cdot \gamma Q \cdot \tau CwWS(0) + \gamma T \cdot \tau CTw(0) = 0.016\ \mathbf{MPa}$$

Core crushing stress

$$LC5\Sigma fCc := \gamma G \cdot fCcwG(0) + \gamma Q \cdot fCcwQ(0) = 0.029\ \mathbf{MPa}$$

Uplift support reaction

$$LC6\Sigma FR := 1 \cdot FRwG(0) + \gamma Q \cdot FRwWS(0) = -2.321\ \mathbf{kN}$$

Resistance checks - ULS

Design stresses and forces

"Outer sheet in compresion"

$EdF1compr := min(LC1\Sigma F1, LC2\Sigma F1, LC3\Sigma F1, LC4\Sigma F1, LC5\Sigma F1, LC6\Sigma F1, LC7\Sigma F1, LC8\Sigma F1)$

$EdF1compr = -128.36\ \mathbf{MPa}$

"Outer sheet in tension"

$EdF1tension := \max(LC1\Sigma F1, LC2\Sigma F1, LC3\Sigma F1, LC4\Sigma F1, LC5\Sigma F1, LC6\Sigma F1, LC7\Sigma F1, LC8\Sigma F1)$

$EdF1tension = 83.2\ \mathbf{MPa}$

"Inner sheet in compression"

$EdF2compr := min(LC1\Sigma F2, LC2\Sigma F2, LC3\Sigma F2, LC4\Sigma F2, LC5\Sigma F2, LC6\Sigma F2, LC7\Sigma F2, LC8\Sigma F2)$

$EdF2compr = -85.23\ \mathbf{MPa}$

"Inner sheet in tension"

$EdF2tension := \max(LC1\Sigma F2, LC2\Sigma F2, LC3\Sigma F2, LC4\Sigma F2, LC5\Sigma F2, LC6\Sigma F2, LC7\Sigma F2, LC8\Sigma F2)$

$EdF2tension = 131.54\ \mathbf{MPa}$

"Core shear"

$Ed\tau C := \max(LC1\Sigma\tau C, LC2\Sigma\tau C, LC3\Sigma\tau C, LC4\Sigma\tau C, LC5\Sigma\tau C, LC6\Sigma\tau C, LC7\Sigma\tau C, LC8\Sigma\tau C)$

$Ed\tau C = 0.03\ \mathbf{MPa}$

"Core crushing at support"

$EdfCc := LC5\Sigma fCc = 0.03\ \mathbf{MPa}$

"Fasteners in uplift"

$EdFRfast := LC6\Sigma FR = -2.32\ \mathbf{kN}$

Design checks

"Outer sheet in compression"

"Strength utilisation" $\dfrac{|EdF1compr|}{RdF1compr} = 0.73$ "ULS check satisfied"

"Outer sheet in tension"

"Strength utilisation" $\dfrac{|EdF1tension|}{RdF1tension} = 0.42$ "ULS check satisfied"

"Inner sheet in compression"

"Strength utilisation" $\dfrac{|EdF2compr|}{RdF2compr} = 0.58$ "ULS check satisfied"

"Inner sheet in tension"

"Strength utilisation" $\dfrac{|EdF2tension|}{RdF2tension} = 0.66$ "ULS check satisfied"

"Core shear"

"Strength utilisation" $\dfrac{|Ed\tau C|}{Rd\tau C} = 0.32$ "ULS check satisfied"

"Core crushing"

"Strength utilisation" $\dfrac{|EdfCc|}{RdfCc} = 0.33$ "ULS check satisfied"

"Fasteners in uplift" $\dfrac{|EdfFRfast|}{RdFRfast} = 0.43$ "ULS check satisfied"

Deflection checks - SLS

Load combinations - SLS

$$LC9\Sigma\Delta := 1 \cdot \Delta wG\left(\frac{1}{2}\right) + 1 \cdot \Delta wQ\left(\frac{1}{2}\right) + \Delta Ts\left(\frac{1}{2}\right) = 33.32\ \mathbf{mm}$$

$$LC10\Sigma\Delta := 1 \cdot \Delta wG\left(\frac{1}{2}\right) + 1 \cdot \Delta wQ\left(\frac{1}{2}\right) + \Delta Tw\left(\frac{1}{2}\right) = 33.44\ \mathbf{mm}$$

$$LC11\Sigma\Delta := 0.75 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot \Delta wQ\left(\frac{1}{2}\right) + 0.6 \cdot \Delta Ts\left(\frac{1}{2}\right) = 25\ \mathbf{mm}$$

$$LC12\Sigma\Delta := 0.75 \cdot 0.6 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot 0.6 \cdot \Delta wQ\left(\frac{1}{2}\right) + 1 \cdot \Delta Ts\left(\frac{1}{2}\right) = 14.96\ \mathbf{mm}$$

$$LC13\Sigma\Delta := 0.75 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot \Delta wQ\left(\frac{1}{2}\right) + 0.6 \cdot \Delta Tw\left(\frac{1}{2}\right) = 25.07\ \mathbf{mm}$$

$$LC14\Sigma\Delta := 0.75 \cdot 0.6 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot 0.6 \cdot \Delta wQ\left(\frac{1}{2}\right) + 1 \cdot \Delta Tw\left(\frac{1}{2}\right) = 15.07\ \mathbf{mm}$$

$$LC15\Sigma\Delta := 1 \cdot \Delta wG\left(\frac{1}{2}\right) + \Delta wWS\left(\frac{1}{2}\right) + \Delta Ts\left(\frac{1}{2}\right) = -19.35\ \mathbf{mm}$$

$$LC16\Sigma\Delta := 1 \cdot \Delta wG\left(\frac{1}{2}\right) + \Delta wWS\left(\frac{1}{2}\right) + \Delta Tw\left(\frac{1}{2}\right) = -19.23\ \mathbf{mm}$$

$$LC17\Sigma\Delta := 0.75 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot \Delta wWS\left(\frac{1}{2}\right) + 0.6 \cdot \Delta Ts\left(\frac{1}{2}\right) = -14.5\ \mathbf{mm}$$

$$LC18\Sigma\Delta := 0.75 \cdot 0.6 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot 0.6 \cdot \Delta wWS\left(\frac{1}{2}\right) + 1 \cdot \Delta Ts\left(\frac{1}{2}\right) = -8.74\ \mathbf{mm}$$

$$LC19\Sigma\Delta := 0.75 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot \Delta wWS\left(\frac{1}{2}\right) + 0.6 \cdot \Delta Tw\left(\frac{1}{2}\right) = -14.43\ \mathbf{mm}$$

$$LC20\Sigma\Delta := 0.75 \cdot 0.6 \cdot \Delta wG\left(\frac{1}{2}\right) + 0.75 \cdot 0.6 \cdot \Delta wWS\left(\frac{1}{2}\right) + 1 \cdot \Delta Tw\left(\frac{1}{2}\right) = -8.63\ \mathbf{mm}$$

Design deflections - SLS

$\Sigma\Delta pressure := \max(LC9\Sigma\Delta, LC10\Sigma\Delta, LC11\Sigma\Delta, LC12\Sigma\Delta, LC13\Sigma\Delta, LC14\Sigma\Delta, LC15\Sigma\Delta, LC16\Sigma\Delta, LC17\Sigma\Delta, LC18\Sigma\Delta, LC19\Sigma\Delta, LC20\Sigma\Delta)$

$$\Sigma\Delta pressure = 33.439 \; \boldsymbol{mm}$$

$\Sigma\Delta uplift := min(LC9\Sigma\Delta, LC10\Sigma\Delta, LC11\Sigma\Delta, LC12\Sigma\Delta, LC13\Sigma\Delta, LC14\Sigma\Delta, LC15\Sigma\Delta, LC16\Sigma\Delta, LC17\Sigma\Delta, LC18\Sigma\Delta, LC19\Sigma\Delta, LC20\Sigma\Delta)$

$$\Sigma\Delta uplift = -19.349 \; \boldsymbol{mm}$$

Design checks - SLS

"Deflection utilisation" $\quad \dfrac{\Sigma\Delta pressure}{\Delta pressureLimit} = 1 \quad$ "SLS check satisfied"

$\dfrac{|\Sigma\Delta uplift|}{\Delta upliftLimit} = 0.44 \quad$ "SLS check satisfied"

Appendix B Actions on buildings

A summary of the actions and the characteristic load magnitudes applied to the generic structures are presented in Table B.1.

Table B.1 Actions and characteristic loading on buildings

Action	Loading			Comments
	Small building	Medium building	Large building	
Roof cladding	0.13kN/m^2			For modern sandwich panel, U=0.15W/m^{2}K
Purlins	0.02 – 0.05kN/m^2			Depending on frame spacing. Based on manufacturers' data
Services	0.20kN/m^2			BS EN 1991-1-1:2002, UK NA
Imposed	0.60kN/m^2			BS EN 1991-1-1:2002, UK NA
Snow	0.40kN/m^2			BS EN 1991-1-3 :2003, UK NA, Oxford
Wind	+0.21kN/m^2 roof pressure -0.55/-0.41kN/m^2 roof suction +0.69kN/m^2 wall pressure -0.34kN/m^2 wall suction	+0.24kN/m^2 roof pressure -0.66/-0.50kN/m^2 roof suction +0.83kN/m^2 wall pressure -0.41kN/m^2 wall suction	+0.24kN/m^2 roof pressure -0.78/-0.39kN/m^2 roof suction +0.63kN/m^2 wall pressure -0.47kN/m^2 wall suction	BS EN 1991-1-4:2005, UK NA, Oxford

There was no inclusion of accidental actions, crane loading and provisions for robustness.

B.1 Permanent load

The permanent loads consist of the weights of the frame, purlins (if present), cladding and services. The weight of the purlins depends on the sections used to accommodate the various spans and spacing between frames and reference to manufacture's datasheets was made (METSEC, 2011). The weight of the roof cladding was assumed equal to 0.13kN/m^2, corresponding to a modern profiled sandwich panel of 135mm PIR insulation with a U-value of 0.15W/m^{2}K (Appendix A) to accommodate anticipated future requirements. The weights from wall cladding, eaves and tie struts were not taken into account since their contribution is very small in design. The services load was taken equal to 0.20kN/m^2 which is typical for the buildings under consideration. Allowance for weight from photovoltaic panels on the roofs was not made due to variations in systems weights. Furthermore, their impact on the frame design is not generally significant (Moutaftsis and Heywood, 2012b). Hence, for the comparative purpose of the study, weights from photovoltaic modules on roofs were not taken into account.

B.2 Imposed Load

The imposed load used was equal to 0.60kN/m^2 according to BS EN 1991-1-1:2005 and the UK National Annex for the roof being accessible only for maintenance (Type H), which is the typical case for such building types.

B.3 Snow load

For the snow load magnitude, it was assumed that the buildings are located in Oxford, UK and the calculation was performed according to BS EN 1991-1-3:2003 and the UK National Annex considering uniform load. The asymmetric load case was not included in the analysis. Drifted snow is considered an accidental action according to the UK National Annex, hence not included in the analysis either.

Following BS EN 1991-1-3 and the UK National Annex the snow load was:

$s = \mu_i \times C_e \times C_t \times s_k = 0.40kN/m^2$

Where:

$\mu_i = 0.8$, is the snow load coefficient for duo-pitch roof below 30^o

$C_e = 1$, is the exposure coefficient for normal topography

$C_t = 1$, is the thermal coefficient

$s_k = 0.5$, is the characteristic value of snow load on the ground at Oxford

B.4 Wind load

For the wind load magnitudes, it was assumed that the buildings are located in Oxford, UK and the calculation was performed according to BS EN 1991-1-4:2005 and the UK National Annex. The building orientation was non-specific. Wind increase at the edges of the roof was ignored in order to simplify the study and due to the fact that its implication to the frame design is small (it primarily influences the cladding arrangement). Furthermore, a uniform pressure across each building's wall and roof was assumed by ignoring the high wind pressure zones. This was based on the assumption that all frames within each building were the same in order to simplify the analysis. This is a typical approach for consulting engineering practice.

The gable frames would normally be susceptible to higher wind pressure. However, they are often over-designed because they resist load from reduced area and possess wind bracing and posts. Hence, only the loads from at the intermediate wind zones of the buildings were considered.

B.5 Load combinations

The action combination expression used is shown in Table B.2. The partial and combination factors used are shown in Table B.3.

Table B.2 Expression for combinations of actions (from BS EN 1990:2002)

Expression	**Permanent actions**		**Variable actions**	
	Unfavourable	Favourable	Leading	Accompanying
6.10 (BS EN 1990:2002)	$\gamma_{Gj,sup}\ G_{kj,sup}$	$\gamma_{Gj,inf}\ G_{kj,inf}$	$\gamma_{Q,1}\ Q_{k,1}$	$\gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$

Table B.3 Partial and combination factors (from BS EN 1990:2002)

Factor	**Value**	**Use**
$\gamma_{Gj,sup}$	1.35	Partial factor for unfavourable permanent actions
$\gamma_{Gj,inf}$	1.00	Partial factor for favourable permanent actions
$\gamma_{Q,i}$	1.50	Partial factor for unfavourable variable actions
ψ_0	0.50	Combination factor for wind actions and snow loads for site altitude below 1000m above sea level
	0.70	Combination factor for imposed roof loads

The Load Cases (LCs) applied to the frames are given below.

For ULS the following load combinations were applied:

- LC1: (1.35) x Permanent load + (1.50) x Imposed load
- LC2: (1.35) x Permanent load + (1.50) x Wind pressure (leading) + (0.5 x 1.50) x Snow load
- LC3: (1.35) x Permanent load + (1.50) x Snow (leading) + (0.5 x 1.50) x Wind pressure load

- LC4: (1.00) x Permanent load + (1.50) x (0.5 x 1.50) x Wind suction load

For SLS the following load combinations were applied, based on the guidance by SCI (2010):

- LC5: Wind load
- LC6: Snow load
- LC7: Imposed load
- LC8: 80% of Wind plus Snow load

According to BS EN 1990:2002, there is no need to include deflections from permanent actions in the SLS checks.

Appendix C Buildings with long span roof envelope systems

C.1 Modelling input and assumptions

C.1.1 Schemes 1, 2 and 3

For the design of the portal frames (Schemes 1, 2 and 3), the following input assumptions were made:

- Frames are elastically designed. Nonlinear effects were taken into account.
- Members are made of Universal Beam sections.
- The haunches are cuts from the rafter sections and their lengths are equal to 1/10 of the bay's span.
- Bases are nominally pinned.
- The outer flange of the rafter is fully restrained for lateral torsional buckling; restraint is provided either by the presence of purlins (Scheme 1) or by the cladding fastened directly to the rafters (Schemes 2 and 3).
- The lower flange of the rafter is restrained at certain locations against lateral torsional buckling when in compression (uplift case); torsional restraint is provided either by 'stays' (when purlins are present – Scheme 1) or by tie-struts (when no purlins are used – Schemes 2 and 3). The locations of the bottom flange restraints for the various cases are shown in Table C.1. Torsional restraints are designed to resist 1% of the maximum value of the factored force in the compression flange within the relevant span according to BS 5950-1:2000.
- Eaves struts are designed to resist horizontal forces equal to 0.5% of the factored load applied on the roof.
- The columns were assumed to be effectively restrained against lateral torsional buckling with addition of torsional restraints.
- Wall and roof bracing members were designed to resist both tension and compression.
- Steel grade is S355.
- Deflection limits for SLS are according to SCI Advisory Note AD090 (SCI, 2010) and are shown in Table C.2.
- Differential deflections between adjacent frames were not taken into account.

- The number of bays for each building size and structural scheme is as previously shown in Table 4.2 and Table 4.3.
- Purlin sizing was performed according to manufacturer's load-span tables (METSEC, 2011) and assumed double span conditions with heavy end bay connections.

Table C.1 Restraint locations of bottom flange for portal frames

Portal frame type	Building size / number of bays	Rafter's length (m)	Restraint locations of bottom flange – distance from eaves (m)
Duo-pitch	Small (1-bay)	12.57	0, 1.80, 3.60, 10.80, 12.50
	Medium (1-bay)	25.13	0, 1.80, 3.60, 5.40, 12.60, 19.80, 23.40, 25.00
	Medium (2-bay)	12.57	0, 1.80, 2.60, 10.80, 12.50
	Large (2-bay)	20.11	0, 1.80, 3.60, 4.50, 18.00, 19.80
Re-oriented	All sizes (multi-bay)	20.00	0, 1.00, 2.00, 3.00, 6.66, 10.00, 13.33, 17.00, 18.00, 19.00, 20.00

Table C.2 Deflection limits for portal frames (according to SCI Advisory Note AD090)

Type of cladding	Deflection limit
Profiled metal sheeting	Vertical: No restriction Horizontal: Height to eaves / 100

C.1.2 Scheme 4

For the design of the trussed roof frames (Scheme 4), the following input assumptions were made:

- Frames are elastically designed. Nonlinear effects were activated for the analysis.
- The truss ends are laying on the column tops by nominally pinned connections.
- Bases are nominally fixed.
- Circular hollow sections are used for the chords and webs.
- The upper and lower chords are restrained for in-plane buckling at the points of intersections with the webs (vertical and diagonals).
- The upper and lower chords of the truss are considered to be fully restrained for out-of-plane buckling; restraint is provided by the cladding which is fastened directly on to the chords.
- The transverse trusses are designed to resist 1% of the maximum value of the factored force in the compression chord within the relevant span of the main truss according to BS 5950-1:2000. Each one is considered to comprise a pinned connection at the intersection with the main truss upper chord and roller connections at the intersections with the main truss lower chords.

- Transverse trusses at eaves are designed to resist horizontal forces equal to 0.5% of the factored load applied on the roof.
- The web members of truss are designed for a buckling length factor equal to 0.9 for in-plane buckling and a buckling length factor of 1.0 for out-of-plane buckling (AccessSteel, 2006)
- The columns were assumed to be properly restrained against lateral torsional buckling with addition of torsional restraints.
- Wall and roof bracing members were designed to resist both tension and compression.
- Steel grade is S355.
- Deflection limits are according to BS EN 1993-1-3:2005 and the UK NA and are shown in Table C.3.
- The number of bays for each building size is as previously shown in Table 4.4.

Table C.3 Deflection limits for the trussed roof frames (adopted from BS EN 1993-1-1:2005 and UK National Annex)

Component type	**Deflection limit**
Other beams (except purlins and sheeting rails)	Vertical: Span / 200
Tops of columns in single storey buildings except portal frames	Horizontal: Height / 300

C.2 Summary of design

The current section shows the output of the preliminary design for each of the adopted structural schemes and for each building size. The results are shown in Table C.4 to Table C.18.

C.2.1 Small building design

The results are summarised below.

Table C.4 Member design for duo-pitch portal frames with purlins scheme (Small building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Rafter	UB 406x178x54	UB 356x171x67	UB 457x191x67	UB 533x210x82
Column	UB 457x152x60	UB 406x178x74	UB 533x210x82	UB 533x210x101
Purlins	202 Z 20	232 Z 16	262 Z 16	342 Z 25
Eaves strut	CHS 88.9x3.2	CHS 101.6x3.2	CHS 101.6x4.5	CHS 139x3.2
Roof bracing	CHS 60.3x2.9	CHS 76.1x2.9	CHS 76.1x2.9	CHS 88.9x2.9
Wall bracing	CHS 101.6x3.2	CHS 114.3x3.2	CHS 114.3x4.5	CHS 139.7x4

Table C.5 Member design for duo-pitch portal frames without purlins scheme (Small building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Rafter	UB 406x178x54	UB 406x178x60	UB 457x191x67	UB 533x210x82
Column	UB 457x152x60	UB 457x191x67	UB 457x191x82	UB 533x210x101
Tie strut	CHS 48.3x2.9	CHS 60.3x2.9	CHS 76.1x2.9	CHS 88.9x2.9
Eaves strut	CHS 88.9x3.2	CHS 101.6x3.2	CHS 101.6x4.5	CHS 139x3.2
Roof bracing	CHS 60.3x2.9	CHS 76.1x2.9	CHS 76.1x2.9	CHS 88.9x2.9
Wall bracing	CHS 101.6x3.2	CHS 114.3x3.2	CHS 114.3x4.5	CHS 139.7x4

Table C.6 Member design for flat-pitch multi-bay re-oriented portal frames scheme (Small building)

Member type	Frame spacing	
	6.25m	**12.50m**
Rafter – Apex	UB 406x140x39	UB 406x178x67
Rafter – Eaves	UB 305x102x28	UB 406x140x39
Rafter - Intermediate	UB 406x140x39	-
Column External - Apex	UB 406x140x39	UB 406x178x67
Column External - Eaves	UB 305x102x28	UB 406x140x39
Column External - Intermediate	UB 356x171x45	-
Column Internal – Apex	UB 203x133x25	UB 305x165x40
Column Internal - Eaves	UB 152x89x16	UB 203x133x25
Column Internal - Intermediate	UB 203x133x25	-
Tie strut	CHS 48.3x2.6	CHS 88.9x2.9
Eaves strut	CHS 76.1x2.9	CHS 88.9x3.6
Roof bracing	CHS 60.3x2.9	CHS 88.9x2.9
Wall bracing	CHS 114.3x3.2	CHS 168.3x5

Table C.7 Member design for trussed roof frames with northlights scheme (Small building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Main truss				
Upper chord	CHS 168.3x3.6	CHS 168.3x4	CHS 168.3x5	CHS 219.1x6
Lower chord	CHS 168.3x5	CHS 193.7x5	CHS 193.7x5	CHS 219.1x5
Vertical webs	CHS 21.3x3.2	CHS 26.9x3.2	CHS 26.9x3.2	CHS 33.7x2.6
Diagonal webs	CHS 76.1x2.9	CHS 88.9x2.5	CHS 88.9x3.2	CHS 114.3x3
Transverse truss				
Upper chord	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)
Lower chord	CHS 114.3x3	CHS 114.3x3	CHS 114.3x3	CHS 114.3x3
Vertical webs	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2
Diagonal webs	CHS 33.7x2.6	CHS 33.7x2.6	CHS 42.4x2.6	CHS 60.3x2.5
Column	UB 178x102x19*	UB 203x102x23	UB 203x102x23*	UB 203x133x30
Roof bracing	CHS 48.3x3.2	CHS 60.3x2.6	CHS 60.3x2.9	CHS 76.1x2.9
Wall bracing	CHS 101.6x3.2	CHS 114.3x3.2	CHS 139.7x3.2	CHS 139.7x5

**SLS is critical*

C.2.2 Medium building design

The results are summarised below.

Table C.8 Member design for 1-bay duo-pitch portal frames with purlins scheme (Medium building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Rafter	UB 610x229x125	UB 762x267x134	UB 762x267x147	UB 838x292x176
Column	UB 762x267x147	UB 762x267x173	UB 914x305x201	UB 1016x305x249
Purlins	172 Z 13	202 Z 20	262 Z 16	342 Z 25
Eaves strut	CHS 168.3x5	CHS 168.3x5.6	CHS 193.7x5	CHS 219x5
Roof bracing	CHS 76.1x2.9	CHS 88.9x2.9	CHS 88.9x2.9	CHS 101.6x3.2
Wall bracing	CHS 139.7x5.6	CHS 168.3x5	CHS 168.3x5.6	CHS 193.7x6.3

Table C.9 Member design for 1-bay duo-pitch portal frames without purlins scheme (Medium building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Rafter	UB 610x229x125	UB 686x254x125	UB 762x267x147	UB 838x292x176
Column	UB 762x267x147	UB 762x267x173	UB 838x292x194	UB 1016x305x222
Tie strut	CHS 76.1x2.9	CHS 76.1x3.2	CHS 88.9x3.6	CHS 114.3x3.2
Eaves strut	CHS 168.3x5	CHS 168.3x5.6	CHS 193.7x5	CHS 219x5
Roof bracing	CHS 76.1x2.9	CHS 88.9x2.9	CHS 88.9x2.9	CHS 101.6x3.2
Wall bracing	CHS 139.7x5.6	CHS 168.3x5	CHS 168.3x5.6	CHS 193.7x6.3

Table C.10 Member design for 2-bay duo-pitch portal frames with purlins scheme (Medium building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Rafter	UB 406x178x67	UB 457x191x67	UB 457x191x82	UB 533x210x92
Column - External	UB 457x152x52	UB 457x152x60*	UB 406x178x74	UB 533x210x92
Column - Internal	UB 254x146x37	UB 305x165x40	UB 356x171x45	UB 356x171x57
Purlins	172 Z 13	202 Z 20	262 Z 16	342 Z 25
Eaves strut	CHS 139.7x3.2	CHS 139.7x3.2	CHS 139.7x5	CHS 168x5
Roof bracing	CHS 48.3x2.9	CHS 60.3x2.6	CHS 60.3x2.9	CHS 76.1x2.9
Wall bracing	CHS 101.6x4	CHS 114.3x3.6	CHS 114.3x5	CHS 139.7x4.5

**SLS is critical*

Table C.11 Member design for 2-bay duo-pitch portal frames without purlins scheme (Medium building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Rafter	UB 356x171x67	UB 457x191x67	UB 457x191x82	UB 533x210x92
Column - External	UB 457x152x52	UB 457x152x60	UB 457x191x67	UB 533x210x92
Column - Internal	UB 254x146x37	UB 305x165x40	UB 356x171x45	UB 305x165x54
Tie strut	CHS 60.3x2.6	CHS 60.3x3.2	CHS 76.3x3.2	CHS 88.9x3.6
Eaves strut	CHS 139.7x3.2	CHS 139.7x3.2	CHS 139.7x5	CHS 168x5
Roof bracing	CHS 48.3x2.9	CHS 60.3x2.6	CHS 60.3x2.9	CHS 76.1x2.9
Wall bracing	CHS 101.6x4	CHS 114.3x3.6	CHS 114.3x5	CHS 139.7x4.5

Table C.12 Member design for flat-pitch multi-bay re-oriented portal frames scheme (Medium building)

Member type	Frame spacing		
	6.25m	**8.33m**	**12.50m**
Rafter – Apex	UB 406x140x39	UB 356x171x51	UB 406x178x67
Rafter – Eaves	UB 203x133x30*	UB 254x146x31*	UB 406x140x39
Rafter - Intermediate	UB 406x140x39 UB 406x140x39 UB 406x140x39	UB 406x140x46 UB 356x171x51	UB 406x178x67
Column External - Apex	UB 457x191x67	UB 457x191x67	UB 457x191x67*
Column External - Eaves	UB 254x146x31*	UB 254x146x37*	UB 356x171x45
Column External - Intermediate	UB 356x171x45 UB 356x171x45 UB 305x165x46	UB 356x171x51 UB 356x171x51*	UB 406x178x67
Column Internal – Apex	UB 457x191x67	UB 457x191x67	UB 457x191x74*
Column Internal - Eaves	UB 203x133x25*	UB 203x133x25	UB 203x133x30
Column Internal - Intermediate	UB 254x146x37 UB 305x165x40 UB 305x165x40	UB 305x165x40 UB 305x165x46*	UB 356x171x57
Tie strut	CHS 48.3x2.6	CHS 60.3x2.9	CHS 88.9x2.9
Eaves strut	CHS 76.1x2.9	CHS 88.9x3.2	CHS 101.6x3.6
Roof bracing	CHS 76.1x2.9	CHS 88.9x2.9	CHS 101.6x3.2
Wall bracing	CHS 168.3x5	CHS 193.7x5	CHS 244.5x5

**SLS is critical*

Table C.13 Member design for 1-bay trussed roof frames with northlights scheme (Medium building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Main truss				
Upper chord	CHS 219.1x6	CHS 244.5x6	CHS 273x6	CHS 244.5x10
Lower chord	CHS 244.5x5	CHS 273x5	CHS 323.9x5	CHS 273x8
Vertical webs	CHS 48.3x2.5	CHS 42.4x2.6	CHS 48.3x2.5	CHS 48.3x3
Diagonal webs	CHS 139.7x3.6	CHS 168.3x3.2	CHS 168.3x3.6	CHS 168.3x5
Transverse truss				
Upper chord	CHS 114.3x3	CHS 114.3x3	CHS 114.3x3	CHS 114.3x3
Lower chord	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)	CHS 114.3x3 (intermediate) CHS 21.3x3.2 (eaves)
Vertical webs	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2
Diagonal webs	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2
Column	UB 254x146x37*	UB 305x165x40	UB 356x171x45	UB 356x171x57
Roof bracing	CHS 60.3x2.9	CHS 76.1x2.9	CHS 76.1x2.9	CHS 88.69x2.9
Wall bracing	CHS 168.3x5	CHS 168.3x5	CHS 193.7x5	CHS 219.1x4.5

**SLS is critical*

Table C.14 Member design for 2-bay trussed roof frames with northlights scheme (Medium building)

Member type	Frame spacing			
	6.67m	**8.00m**	**10.00m**	**13.34m**
Main truss				
Upper chord	CHS 114.3x3	CHS 114.3x3.6	CHS 139.7x3.6	CHS 168.3x3.2
Lower chord	CHS 168.3x3.2	CHS 168.3x3.6	CHS 168.3x5	CHS 193.7x5
Vertical webs	CHS 42.4x2.6	CHS 42.4x2.6	CHS 42.4x2.6	CHS 48.3x2.5
Diagonal webs	CHS 114.3x3	CHS 114.3x3.6	CHS 139.7x3.2	CHS 139.7x3.6
Transverse truss				
Upper chord	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2
Lower chord	CHS 114.3x3	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2
Vertical webs	CHS 21.3x3.2 (intermediate) CHS 26.9x3.2 (eaves)	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2
Diagonal webs	CHS 21.3x3.2 (intermediate) CHS 26.9x3.2 (eaves)	CHS 21.3x3.2	CHS 21.3x3.2	CHS 21.3x3.2
Column External	UB 203x133x25	UB 203x133x25	UB 203x133x30	UB 254x146x31
Column Internal	UB 305x165x40	UB 356x171x45	UB 356x171x51	UB 356x171x67
Roof bracing	CHS 18.3x2.6	CHS 18.3x2.6	CHS 60.3x2.6	CHS 60.3x2.6
Wall bracing	CHS 139.7x3.2	CHS 139.7x4.5	CHS 168.3x5	CHS 168.3x5

C.2.3 Large building design

The results are summarised below.

Table C.15 Member design for 2-bay duo-pitch portal frames with purlins scheme (Large building)

Member type	**Frame spacing**			
	6.67m	**8.33m**	**10.42m**	**12.50m**
Rafter	UB 533x210x101	UB 610x229x113	UB 686x254x125	UB 762x267x134
Column - External	UB 610x229x113	UB 762x267x134	UB 686x254x170	UB 838x292x176
Column - Internal	UB 356x171x45	UB 305x165x54	UB 356x171x67	UB 406x178x74
Purlins	172 Z 13	232 Z 16	262 Z 25	342 Z 25
Eaves strut	CHS 168.3x5.6	CHS 219x4.5	CHS 219.1x5.6	CHS 244x5
Roof bracing	CHS 48.3x3.2	CHS 48.3x3.2	CHS 60.3x3.2	CHS 60.3x4.5
Wall bracing	CHS 139.7x3.2	CHS 139.7x4	CHS 139.7x5	CHS 168.3x5

Table C.16 Member design for 2-bay duo-pitch portal frames without purlins scheme (Large building)

Member type	**Frame spacing**			
	6.67m	**8.33m**	**10.42m**	**12.50m**
Rafter	UB 533x210x101	UB 610x229x113	UB 610x229x125	UB 762x267x134
Column - External	UB 610x229x113	UB 686x254x125	UB 762x267x147	UB 762x267x173
Column - Internal	UB 356x171x45	UB 305x165x54	UB 356x171x67	UB 406x178x74
Tie strut	CHS 60.3x2.9	CHS 76.1x2.9	CHS 88.9x3.6	CHS 101.6x3.2
Eaves strut	CHS 168.3x5.6	CHS 219x4.5	CHS 219.1x5.6	CHS 244x5
Roof bracing	CHS 48.3x3.2	48.3x3.2	CHS 60.3x3.2	CHS 60.3x4.5
Wall bracing	CHS 139.7x3.2	CHS 139.7x4	CHS 139.7x5	CHS 168.3x5

Table C.17 Member design for flat-pitch multi-bay re-oriented portal frames scheme (Large building)

Member type	Frame spacing			
	6.66m	**8.88m**	**10.00m**	**13.33m**
Rafter – Apex	UB 356x171x45	UB 457x152x52	UB 457x152x52	UB 457x191x67
Rafter – Eaves	UB 254x146x31*	UB 356x127x33	UB 254x146x37*	UB 406x140x39
Rafter - Intermediate	UB 356x171x45 UB 356x171x45 UB 406x140x46 UB 406x140x46 UB 406x140x39	UB 356x171x51 UB 457x152x52 UB 457x152x52 UB 457x152x52	UB 457x152x52 UB 457x152x60 UB 457x152x60	UB 457x191x67 UB 457x191x67
Column External - Apex	UB 610x229x113	UB 610x229x113	UB 610x229x113	UB 610x229x113
Column External - Eaves	UB 254x146x31	UB 406x140x39	UB 406x140x39	UB 457x152x52
Column External - Intermediate	UB 356x171x45 UB 356x171x45 UB 305x165x40 UB 457x191x74 UB 533x210x101	UB 406x178x60 UB 356x171x51 UB 457x191x67 UB 533x210x92	UB 406x178x67 UB 356x171x51 UB 533x210x82	UB 457x191x67 UB 457x191x74
Column Internal – Apex	UB 610x229x113	UB 610x229x113	UB 610x229x113	UB 610x229x113
Column Internal - Eaves	UB 203x133x25	UB 203x133x25	UB 203x133x25	UB 254x146x31
Column Internal - Intermediate	UB 254x146x37 UB 305x165x40 UB 305x165x46 UB 457x191x74 UB 533x210x101	UB 305x165x40 UB 356x171x51 UB 457x191x67 UB 533x210x92	UB 305x165x46 UB 406x178x60 UB 533x210x82	UB 356x171x67 UB 457x191x82
Tie strut	CHS 48.3x2.9	CHS 60.3x3.2	CHS 76.1x2.9	CHS 88.9x3.2
Eaves strut	CHS 60.3x3.6	CHS 88.9x2.9	CHS 101.6x3.2	CHS 114.3x4
Roof bracing	CHS 76.1x3.2	CHS 88.9x3.2	CHS 101.6x3.2	CHS 114.3x3.2
Wall bracing	CHS 219.1x4.5	CHS 244.5x5	244.5x5.6	CHS 323.9x5

**SLS is critical*

Table C.18 Member design for trussed roof frames with northlights scheme (Large building)

Member type	Frame spacing			
	6.67m	**8.33m**	**10.42m**	**12.50m**
Main truss				
Upper chord	CHS 139.7x6	CHS 168.3x6.3	CHS 219.1x6	CHS 244.5x6.3
Lower chord	CHS 244.5x5	CHS 273x5	CHS 323.9x5	CHS 323.9x5
Vertical webs	CHS 33.7x2.6	CHS 42.4x2.6	CHS 42.4x2.6	CHS 42.4x2.6
Diagonal webs	CHS 139.7x3.2	CHS 139.7x3.6	CHS 168.3x3.2	CHS 168.3x4
Transverse truss				
Upper chord	CHS 114.3x3 (intermediate) CHS 42.4x2.6 (eaves)	CHS 114.3x3 (intermediate) CHS 33.7x3.2 (eaves)	CHS 114.3x3	CHS 114.3x3 CHS 42.4x2.6
Lower chord	CHS 114.3x3	CHS 114.3x3	CHS 114.3x3 CHS 33.7x3.2	CHS 114.3x3 CHS 114.3x3.2
Vertical webs	CHS 26.9x3.2 (intermediate) CHS 21.3x3.2 (eaves)	CHS 33.7x2.6	CHS 33.7x2.6 (intermediate) CHS 21.3x3.2 (eaves)	CHS 26.9x3.2 (intermediate) CHS 21.3x3.2 (eaves)
Diagonal webs	CHS 33.7x2.6	CHS 42.4x2.6	CHS 48.3x2.5	CHS 60.3x2.5
Column - External	UB 203x133x25	UB 203x133x30	UB 254x146x31	UB 254x146x37
Column - Internal	UB 356x171x51	UB 406x178x60	UB 406x178x74	UB 457x191x82
Roof bracing	CHS 18.3x2.6	CHS 18.3x2.6	CHS 60.3x2.6	CHS 60.3x2.6
Wall bracing	CHS 139.7x3.2	CHS 139.7x4.5	CHS 168.3x5	CHS 168.3x5

C.3 Steelwork quantities

This section provides the summary of the structural steelwork material quantities used for each of the structural schemes and building sizes. The quantities are based on the design results shown in the previous section. Frame spacing references have been assigned in order to group similar and identical distances for facilitation of comparison based on Table 4.5.

The sum and breakdown of the steelwork quantities are illustrated in Figure C.1 to Figure C.15.

C.3.1 Small building

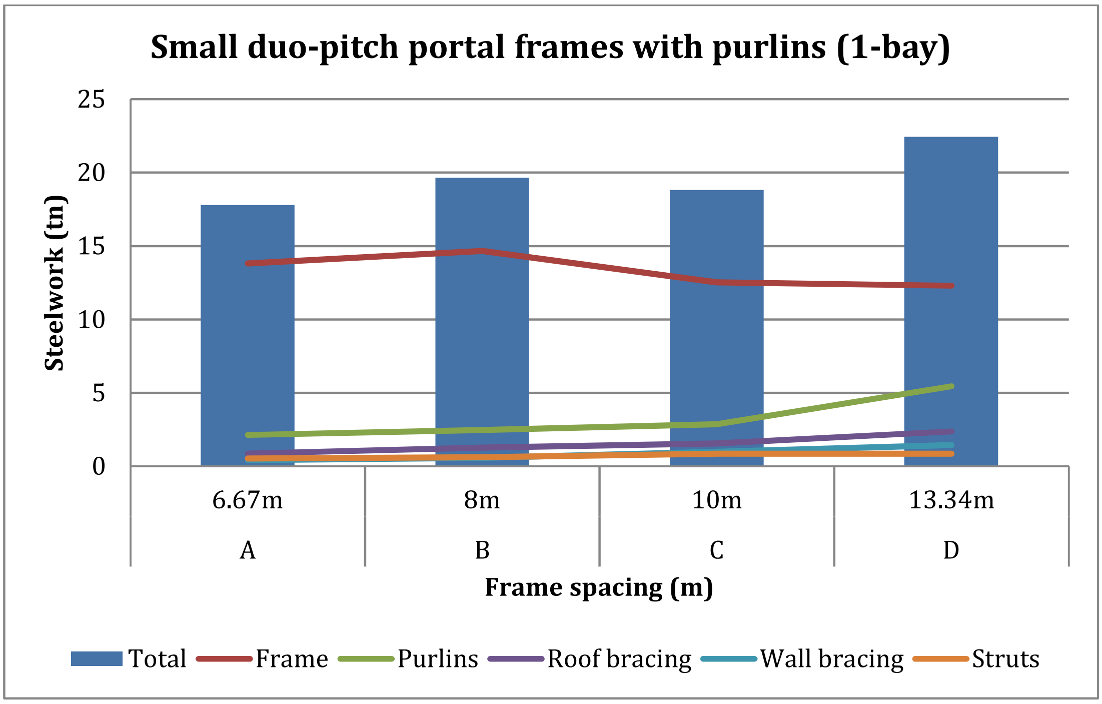

Figure C.1 Steelwork for duo-pitch portal frames with purlins scheme (Small building)

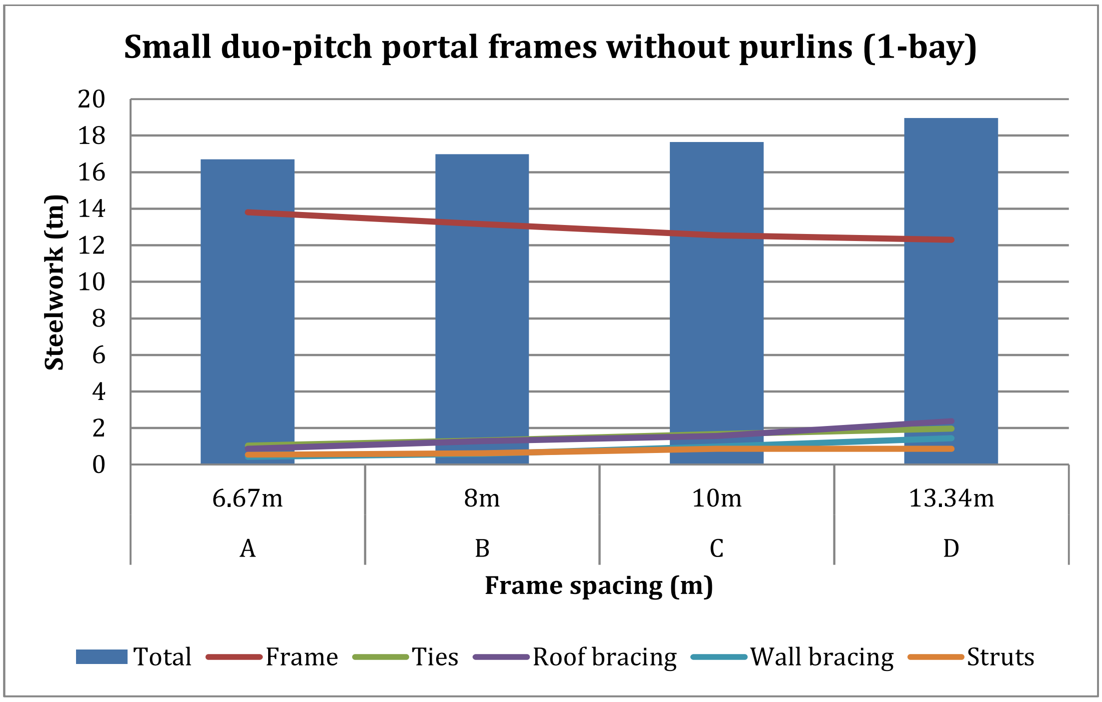

Figure C.2 Steelwork for duo-pitch portal frames without purlins scheme (Small building)

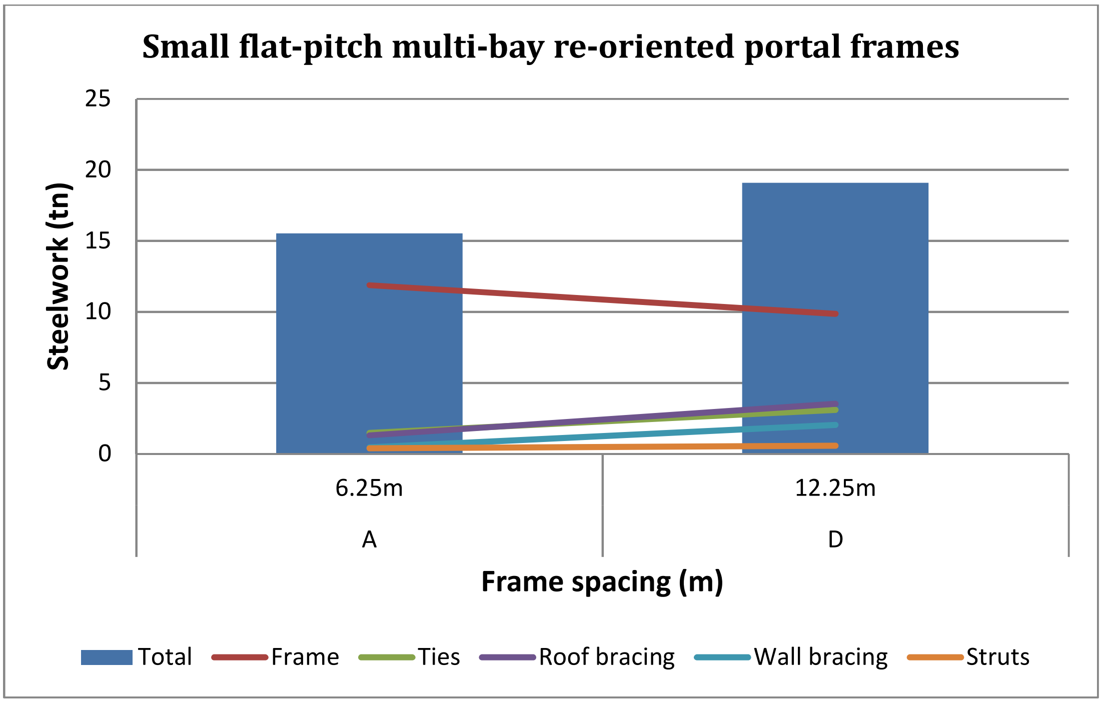

Figure C.3 Steelwork for flat-pitch multi-bay re-oriented portal frames scheme (Small building)

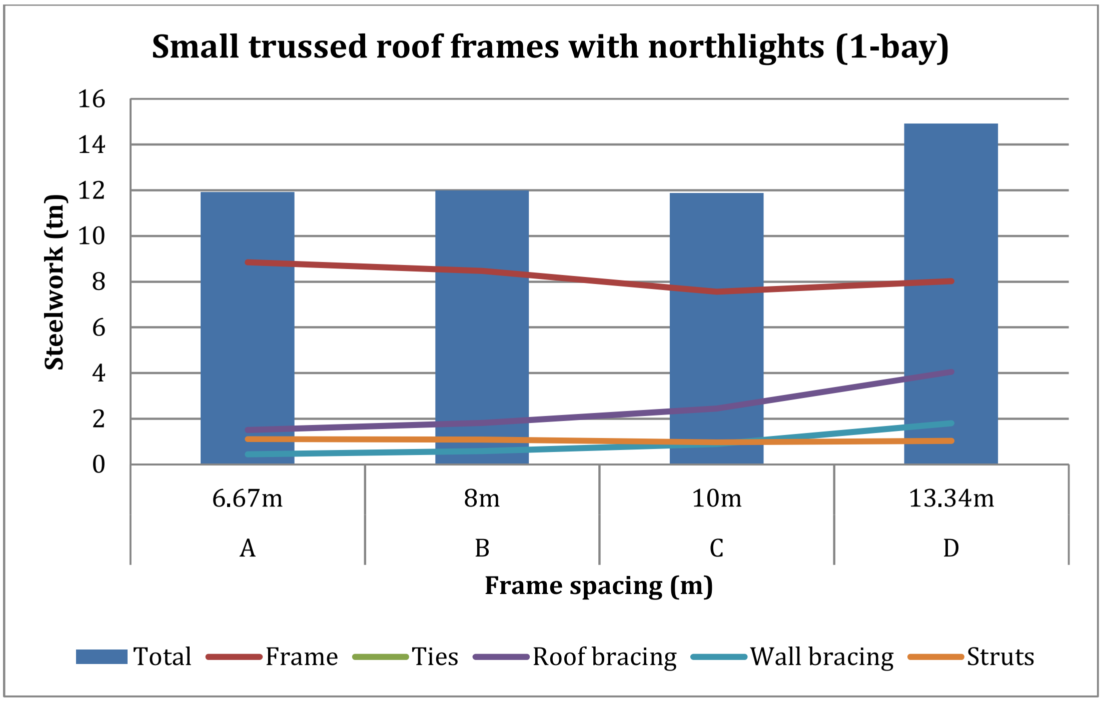

Figure C.4 Steelwork for trussed roof frames with northlights scheme (Small building)

C.3.2 Medium building

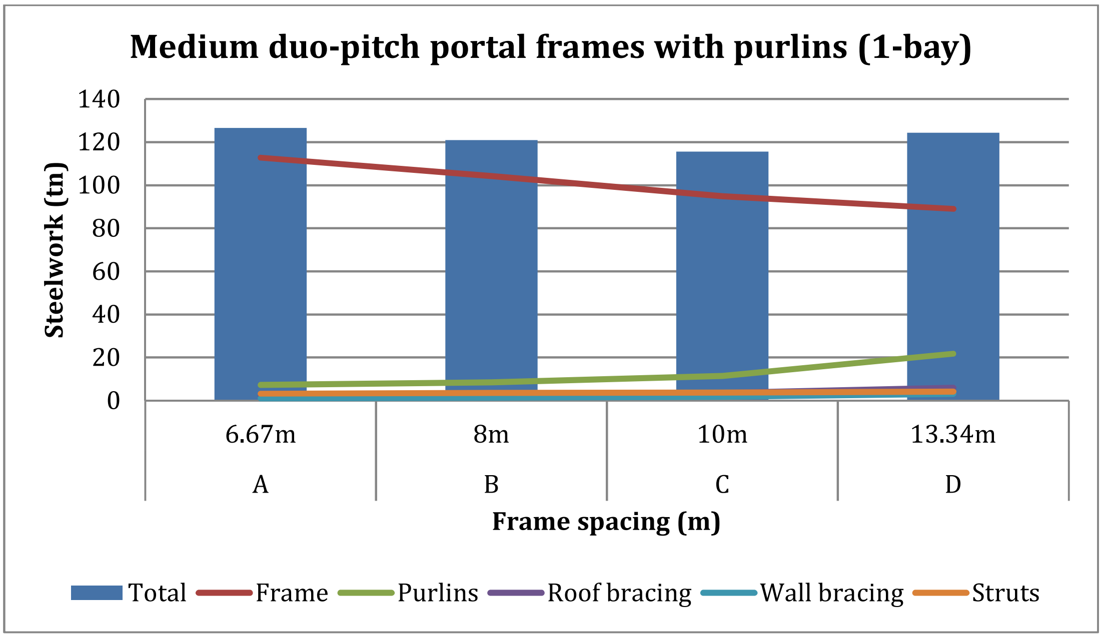

Figure C.5 Steelwork for 1-bay duo-pitch portal frames with purlins scheme (Medium building)

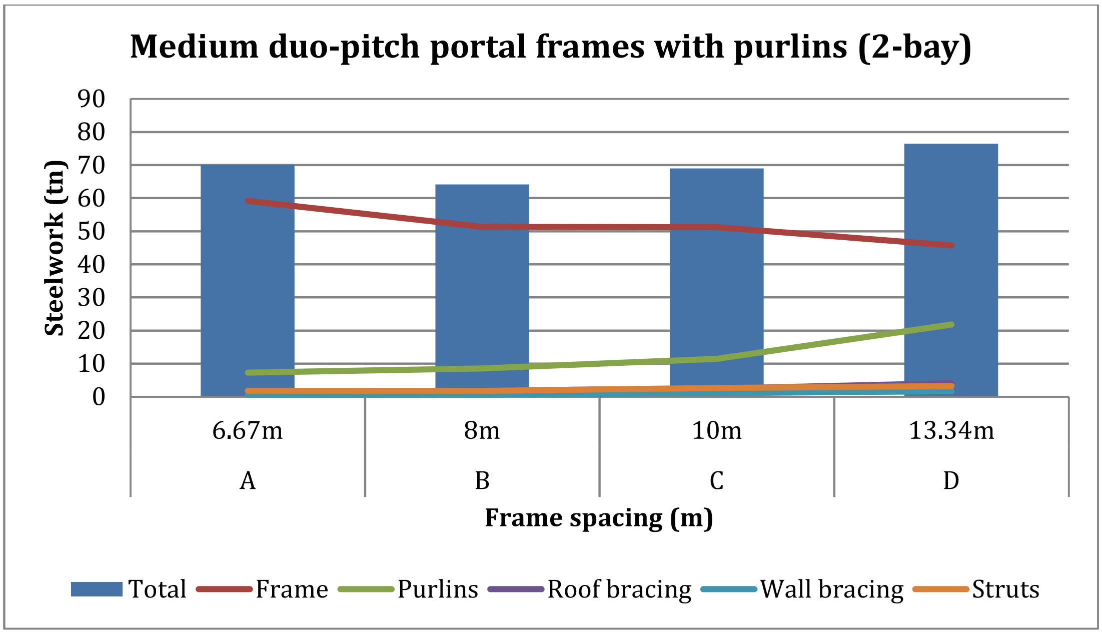

Figure C.6 Steelwork for 2-bay duo-pitch portal frames with purlins scheme (Medium building)

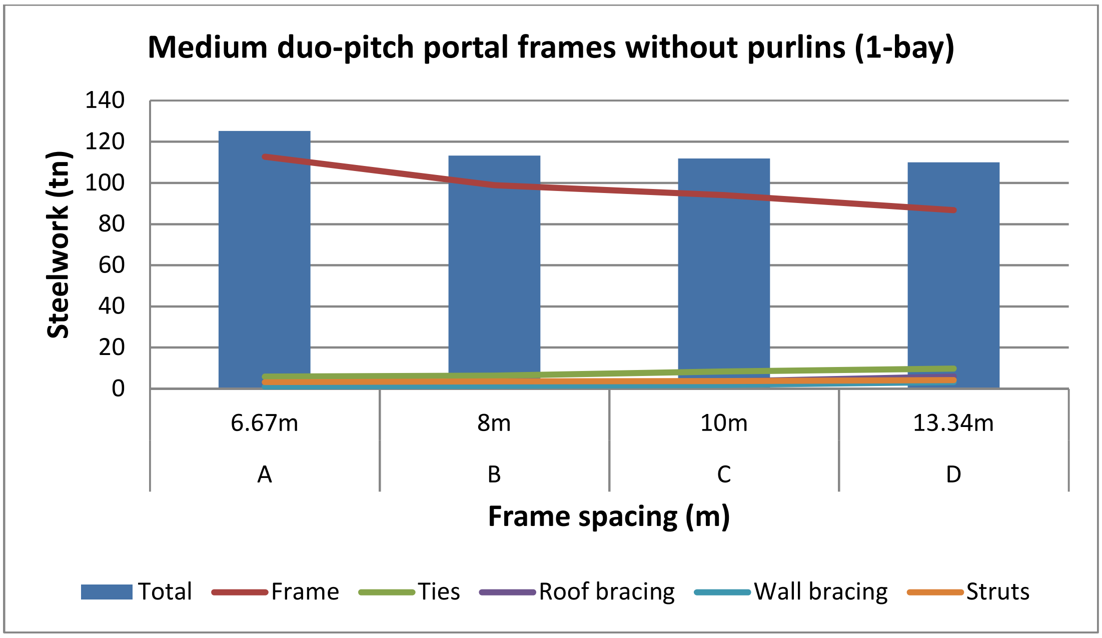

Figure C.7 Steelwork for 1-bay duo-pitch portal frames without purlins scheme (Medium building)

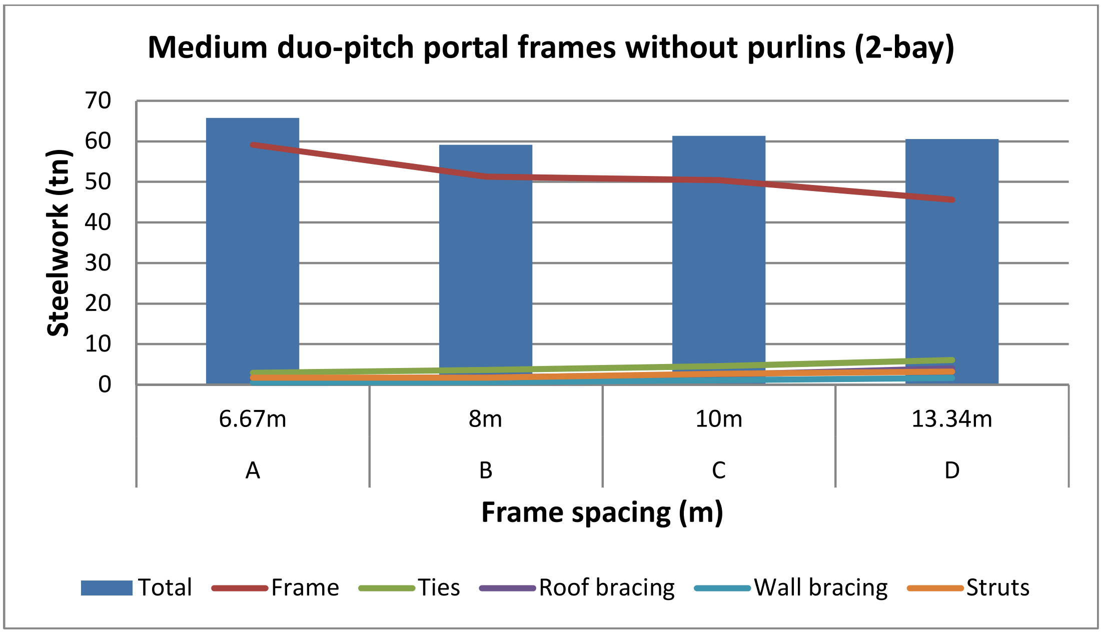

Figure C.8 Steelwork for 2-bay duo-pitch portal frames without purlins scheme (Medium building)

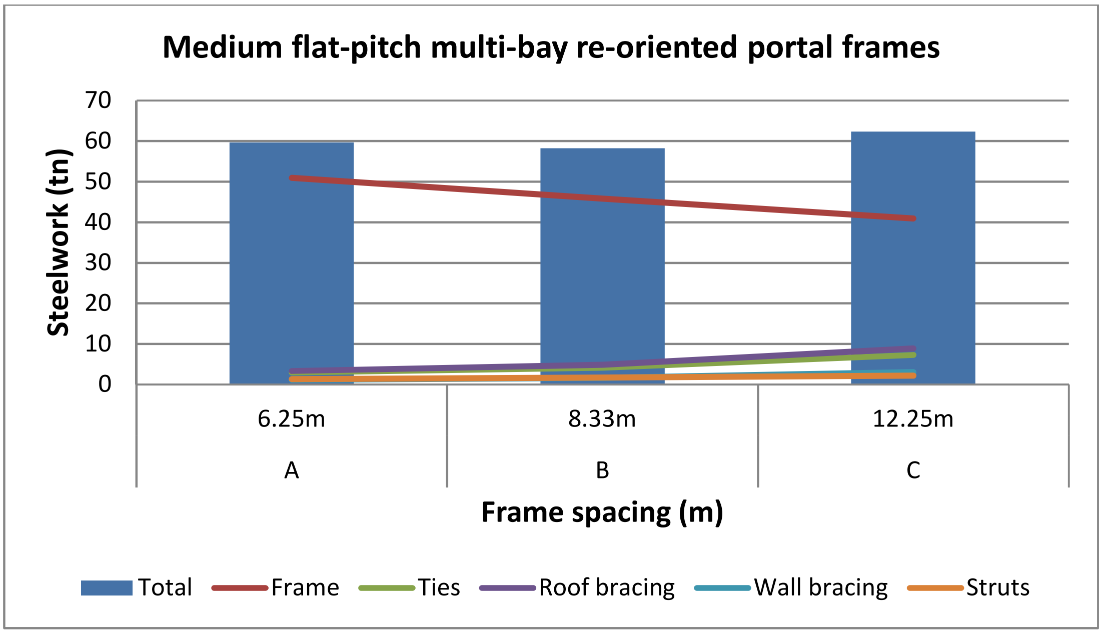

Figure C.9 Steelwork for flat-pitch multi-bay re-oriented portal frames scheme (Medium building

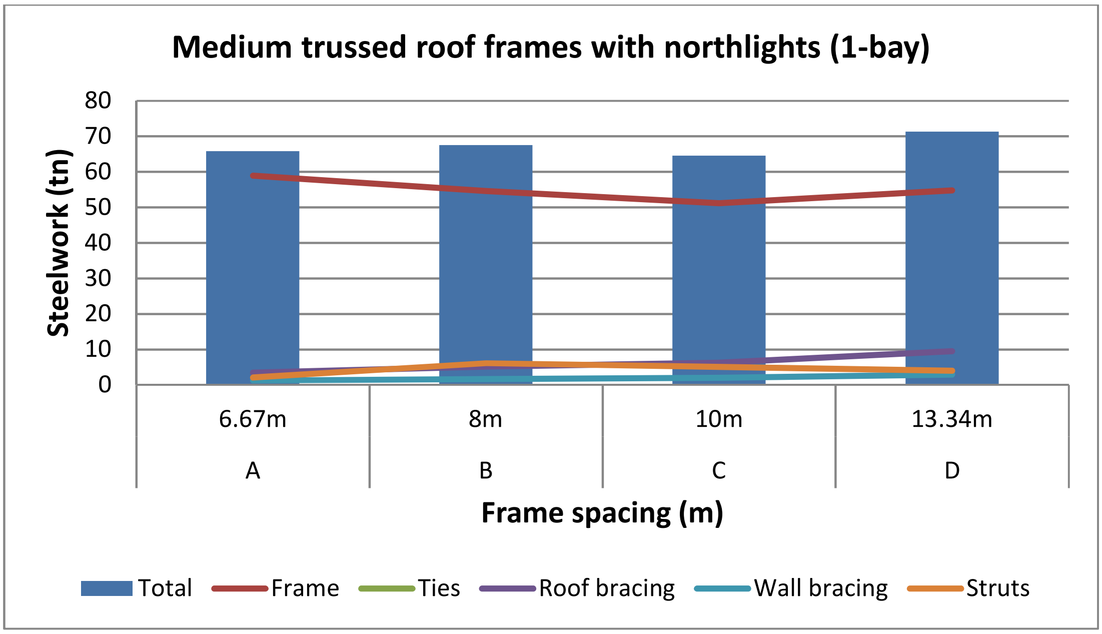

Figure C.10 Steelwork for 1-bay trussed roof frames with northlights scheme (Medium building)

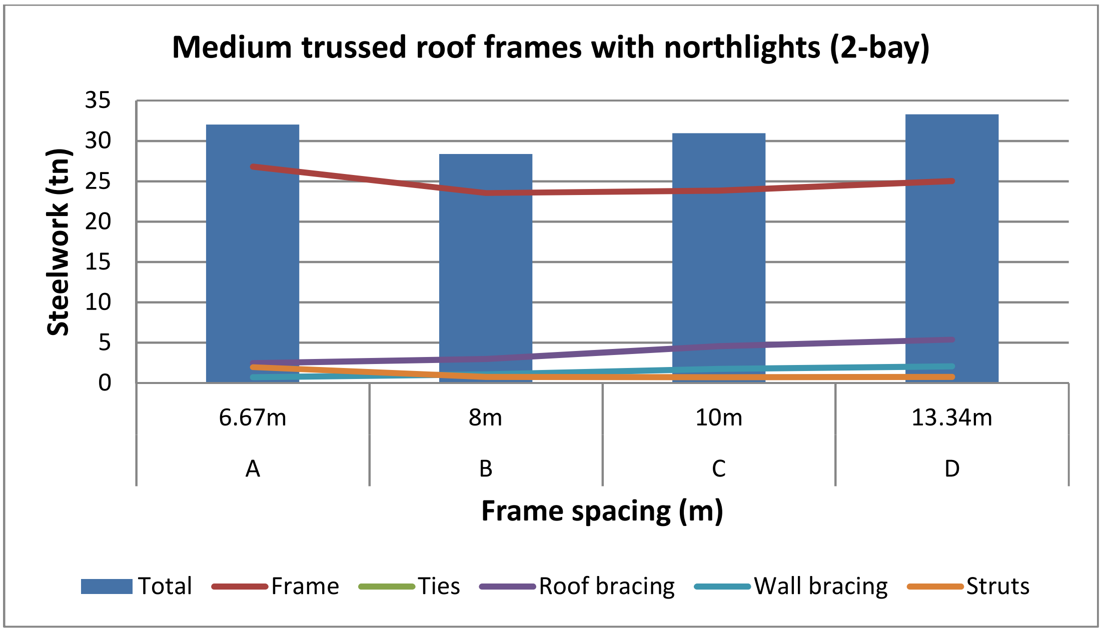

Figure C.11 Steelwork for 2-bay trussed roof frames with northlights scheme (Medium building)

C.3.3 A.3.3 Large building

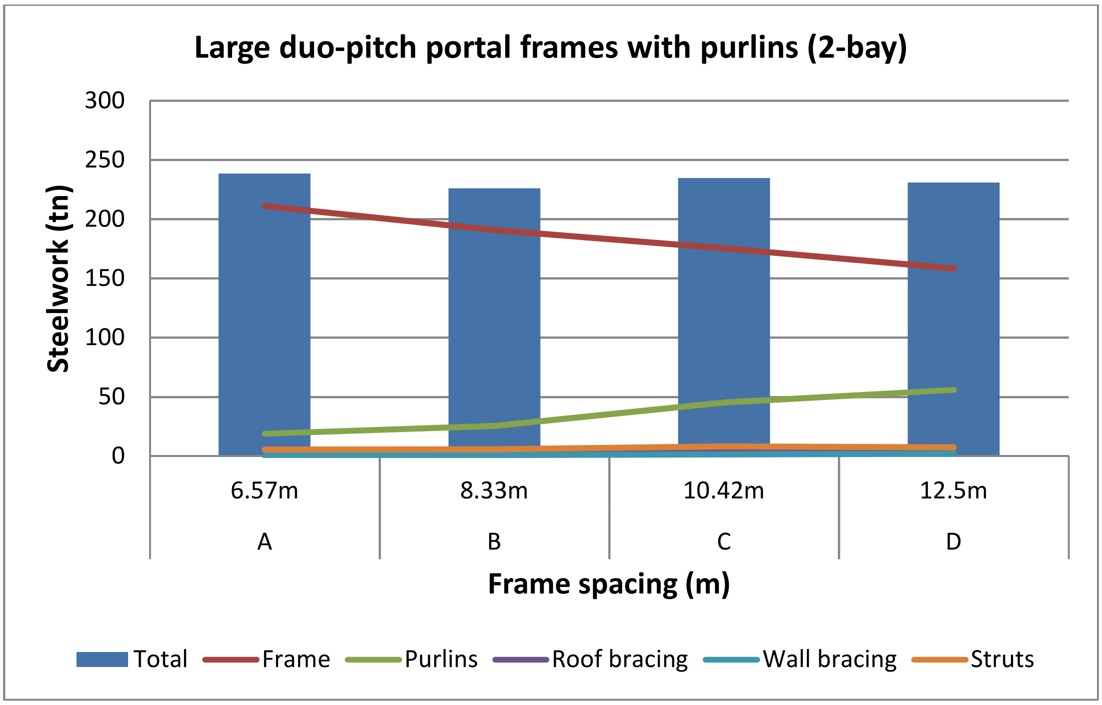

Figure C.12 Steelwork for 2-bay duo-pitch portal frames with purlins scheme (Large building)

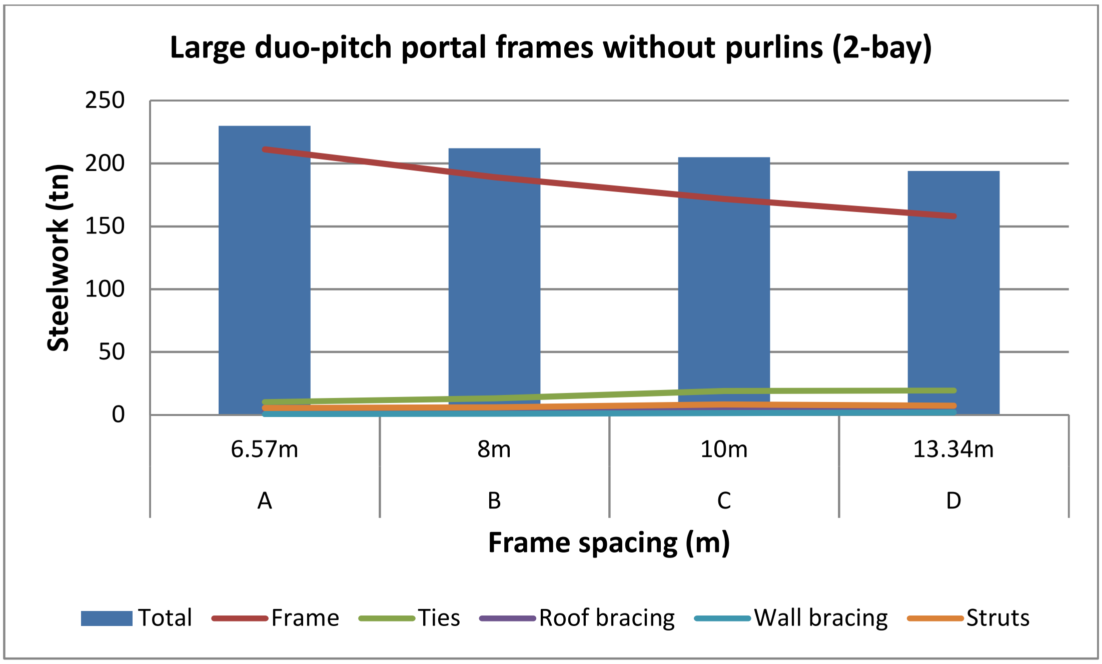

Figure C.13 Steelwork for 2-bay duo-pitch portal frames with purlins scheme (Large building)

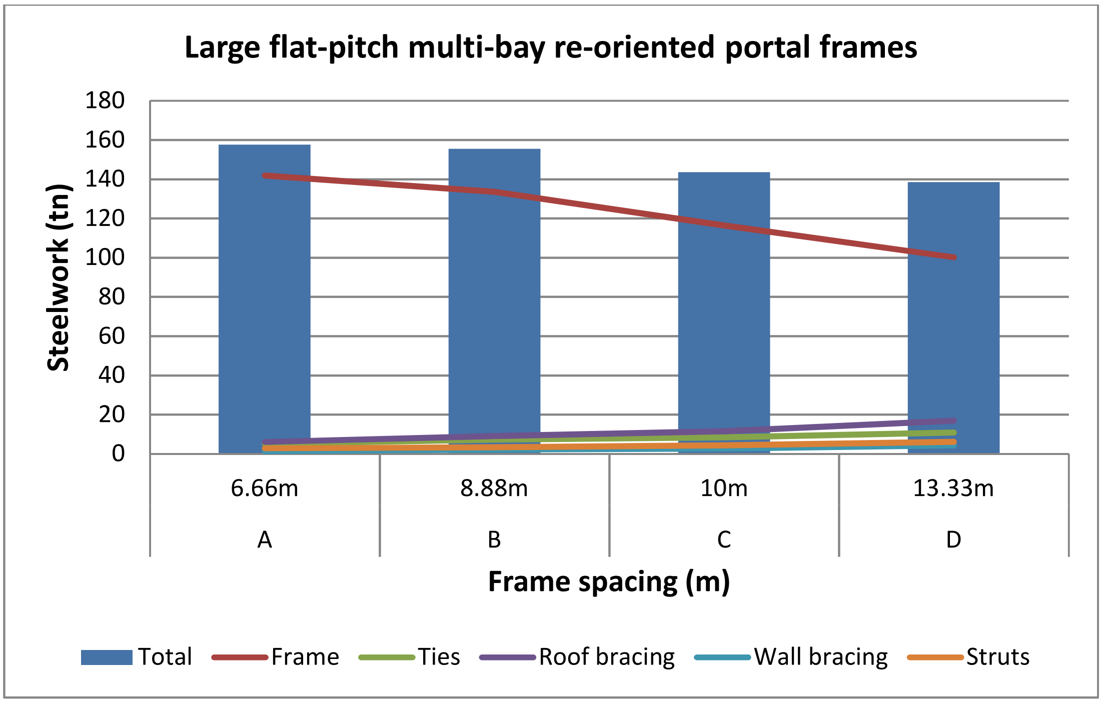

Figure C.14 Steelwork for flat-pitch multi-bay re-oriented portal frames scheme (Large building)

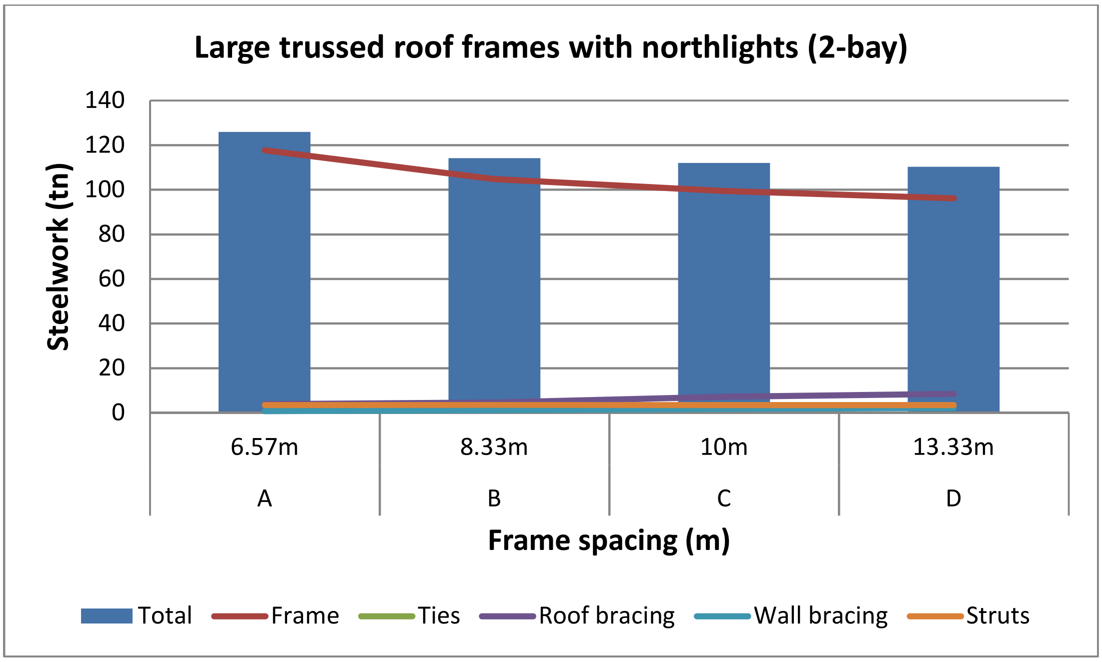

Figure C.15 Steelwork for trussed roof frames with northlights scheme (Large building)

Appendix D Buildings with diaphragm action

D.1 Modelling input and assumptions for shear panels

D.1.1 Sandwich panel systems

For the appraisal of the in-plane resistance and flexibility of sandwich panel roof systems the following input assumptions were made:

- Modelling was based on the fully profiled sandwich panel for roofs shown in Appendix A.
- Fastener resistances and flexibilities in shear were calculated for standard products used in practice (specifications shown in Appendix A) and according to Käpplein and Ummenhofer (2011). These are presented in Table D.1. It should be noted that no discussion is made by Käpplein and Ummenhofer (2011) whether the stiffness of rivets or self-drilling screws for side-lap fastening is influenced by the presence of plastic seal. In practice, the presence of butyl mastic layer at the crests of the side-laps will probably have an onerous influence to the flexibility of the connection.
- Side-lap sealing fasteners were considered at the maximum 450mm spacing according to the sandwich panel manufacturer's guidance for the normal seam fastening case. For the dense seam fastening case the spacing distance was equal to the half of the maximum, i.e. 225mm.
- Fastening to purlins is typically applied at each of the panel's troughs (fastener spacing across the panel's width equal to 333mm) for intermediate support fastening of double- or multi-span arrangements, while the end connections comprise 2 fasteners at each trough (average fastener spacing across the panel's width equal to 167mm).
- The flexibility of shear connectors was assumed to be very low compared to the rest of the fasteners and their resistances higher, hence ignored for the purposes of the feasibility study.
- The calculation of the shear diaphragm capacity was based on the methodology of BS 5950-9:1994 with some modifications to account for sandwich panels, systems which are excluded from that literature. Specifically, shear buckling and end collapse of the profiles and shear flexibility due to profile distortion were

not taken into account since these modes are not met in sandwich panels (Davies and Lawson, 1999).

- The flexibility due to shear strain could be estimated based on the shear modulus of the core, however, there is currently no literature discussing the influence of the aspect ratio of the diaphragm panel. Therefore, it was assumed that the in-plane stiffness of the panel was much higher compared to the connections and, therefore, negligible. The same approach is followed by Käpplein and Misiek (2011a, 2011b, 2011c) and CIB-ECCS (2013).
- The resistances and flexibilities for the various sandwich panel arrangements are shown in Table 5.3 and Table 5.4.

Table D.1 Sandwich panel fastener resistances and flexibilities

Fastening arrangement	Substructure	Model input data	Resistance (kN)	Flexibility (mm/KN)
Panel to sub-structure 5.5mm stainless steel edge fastener with sealing washer (see Appendix A)	Purlin (cold-formed) Flange thickness: 1.4mm	t_{F2}=0.4mm, t_{sup}=1.4mm, f_{uF2}=390MPa, d_1=4.88mm, d_s=5.25mm	0.916	0.386
	Rafter (hot-rolled) Flange thickness: 17.3mm	t_{F2}=0.4mm, t_{sup}=17.3mm, f_{uF2}=390MPa, d_1=4.88mm, d_s=5.25mm	0.916	0.483
		t_{F2}=0.7mm, t_{sup}=17.3mm, f_{uF2}=390MPa, d_1=4.88mm, d_s=5.25mm	2.119	0.209
	Rafter (hot-rolled) Flange thickness: 11.2mm	t_{F2}=0.4mm, t_{sup}=11.2mm, f_{uF2}=390MPa, d_1=4.88mm, d_s=5.25mm	0.916	0.439
		t_{F2}=0.7mm, t_{sup}=11.2mm, f_{uF2}=390MPa, d_1=4.88mm, d_s=5.25mm	2.119	0.170
Seam fastening 4.8mm sealed rivets or self-drilling fasteners with sealing washers (see Appendix A)		t_{F1}=0.5mm, f_{uF1}=403MPa, d=4.8mm	1.001	0.219
		t_{F1}=0.7mm, f_{uF1}=403MPa, d=4.8mm	1.655	0.156

D.1.2 Built-up and decking systems (benchmarking)

Built-up and decking roof cladding systems were assessed and benchmarked against sandwich panels. The following products and components were included in the analysis:

- Build-up system (Trisobuild) (TATA Steel, 2011) comprising:
    - R32 external sheet and
    - LT1000 internal (liner) sheet (conventional) or
    - RL32 internal (liner) sheet (enhanced)
- Deep decking, RoofDek D60 (TATA Steel, 2012)
- Shallow decking, Profil HV18 (Skandek, 2009)

The results are shown in Table D.2. The input and assumptions made for the appraisal of the in-plane resistances and flexibilities of the systems are discussed in Sections C.1.2.1 and C.1.2.2.

Table D.2 Resistances and flexibilities of built-up and decking shear panel arrangements spanning between purlins (Scheme 1) (benchmarking)

Shear panel dimensions – ref. building	**System**	**Resistance and flexibility**			
		Normal fastening		**Dense fastening**	
		4-sides	**2-sides**	**4-sides**	**2-sides**
12.6m x 6.67m (Small, 1-bay; Medium 2-bay)	*R32 - external*	*34.4kN ** 0.328mm/kN*	*34.4kN ** 0.790mm/kN*	*57.8kN * 0.159mm/kN*	*57.4kN ** 0.620mm/kN*
	LT1000 - liner	*16.3kN ** 0.423mm/kN*	*16.3kN ** 0.788mm/kN*	*24.4kN *** 0.143mm/kN*	*24.4kN *** 0.504mm/kN*
	RL32 - liner	*34.4kN ** 0.294mm/kN*	*34.4kN ** 0.658mm/kN*	*50.8kN * 0.135mm/kN*	*50.8kN * 0.496mm/kN*
	R32 - LT1000	Min 16.3kN Max 50.7kN 0.185mm/kN	Min 16.3kN Max 50.7kN 0.395mm/kN	Min 24.4kN Max 82.2kN 0.075mm/kN	Min 24.4kN Max 81.8kN 0.278mm/kN
	R32 - RL32	Min 34.4kN Max 68.8kN 0.155mm/kN	Min 34.4kN Max 68.8kN 0.359mm/kN	Min 50.8kN Max 108.6kN 0.073mm/kN	Min 50.8kN Max 108.2kN 0.275mm/kN
	Profil HV18	45.6kN ** 0.116mm/kN	45.6kN ** 0.475mm/kN	75.4kN ** 0.074mm/kN	75.4kN ** 0.430mm/kN
	RoofDek D60	34.4kN ** 0.432mm/kN	34.4kN ** 0.794mm/kN	70.3kN * 0.129mm/kN	70.3kN * 0.487mm/kN
25.2m x 6.67m (Medium, 1-bay)	*R32 - external*	*68.9kN ** 0.130mm/kN*	*68.9kN ** 0.375mm/kN*	*113.5kN * 0.070mm/kN*	*113.5kN * 0.304mm/kN*
	LT1000 - liner	*32.6kN ** 0.163m/kN*	*32.6kN ** 0.357mm/kN*	*48.7kN *** 0.061mm/kN*	*48.7kN *** 0.254mm/kN*
	RL32 - liner	*68.9kN ** 0.116mm/kN*	*68.9kN ** 0.310mm/kN*	*117.3kN * 0.054mm/kN*	*167.7kN * 0.246mm/kN*
	R32 - LT1000	Min 32.6kN Max 101.5kN 0.072mm/kN	Min=32.6kN Max=101.5kN 0.188mm/kN	Min=48.7kN Max=162.2kN 0.032mm/kN	Min=48.7kN Max=162.2kN 0.138mm/kN
	R32 - RL32	Min=68.9kN Max=137.8kN 0.061mm/kN	Min=68.9kN Max=137.8kN 0.169mm/kN	Min=113.5kN Max=230.8kN 0.030mm/kN	Min=113.5kN Max=281.2kN 0.136mm/kN
	Profil HV18	91.2kN ** 0.042mm/kN	91.2kN ** 0.235mm/kN	146.6kN * 0.032mm/kN	146.6kN * 0.223mm/kN
	RoofDek D60	68.9kN ** 0.163mm/kN	68.9kN ** 0.342mm/kN	136.9kN * 0.054mm/kN	136.9kN * 0.246mm/kN
20.1m x 6.58m (Large, 2-bay)	*R32 - external*	*55.0N ** 0.185mm/kN*	*55.0kN ** 0.492mm/kN*	*89.6kN * 0.093mm/kN*	*89.6kN * 0.386mm/kN*
	LT1000 - liner	*26.0kN ** 0.238m/kN*	*26.0kN ** 0.481m/kN*	*37.7kN *** 0.083mm/kN*	*37.7kN *** 0.324mm/kN*
	RL32 - liner	*55.0kN ** 0.167mm/kN*	*55.0kN ** 0.409mm/kN*	*92.7kN * 0.071mm/kN*	*92.7kN * 0.312mm/kN*
	R32 - LT1000	Min 26.0kN Max 81.0kN 0.104mm/kN	Min 26.0kN Max 81.0kN 0.243mm/kN	Min 37.7kN Max=127.3kN 0.044mm/kN	Min=37.7kN Max=127.3kN 0.176mm/kN
	R32 - RL32	Min=55.0kN Max=110.0kN 0.088mm/kN	Min=55.0kN Max=110.0kN 0.223mm/kN	Min=89.6kN Max=230.8kN 0.040mm/kN	Min=89.6kN Max=281.2kN 0.172mm/kN
	Profil HV18	72.8kN ** 0.057mm/kN	72.8kN ** 0.297mm/kN	116.1kN * 0.042mm/kN	116.1kN * 0.280mm/kN
	RoofDek D60	55.0kN ** 0.232m/kN	55.0kN ** 0.460mm/kN	108.3kN * 0.072mm/kN	108.3kN * 0.313mm/kN

**Seam failure; **End collapse of sheeting profile; ***Shear buckling*

D.1.2.1 Built-up system modelling details

For the built-up assemblies, the system's stiffness is theoretically calculated assuming two springs in parallel, where every spring reflects the stiffness of each sheeting layer. This theoretical estimation was also previously used by Davies and Lawson (1999); it should be noted that Davies and Lawson (1999) showed variations between the theoretical stiffness and the tested performance of built-up assemblies. Furthermore, the assumption neglects the flexibility of the bar and bracket systems which is used within the built-up assembly. Therefore, all results are indicative. A synopsis for the resistance and stiffness calculation model for two springs in parallel is given in Table D.3.

Table D.3 Resistance and stiffness calculations for two springs in parallel

Equations	**Notation**
$F = F_1 + F_2$	F – resistance
$x = x_1 = x_2$	x – displacement
$k \cdot x = k_1 \cdot x_1 + k_2 \cdot x_2 = x \cdot (k_1 + k_2)$	k – stiffness
$k = k_1 + k_2$	c – flexibility
$1/c = 1/c_1 + 1/c_2 = (c_2 + c_1)/(c_1\ c_2)$	
$c = c_1 \cdot c_2 / (c_1 + c_2)$	

The following input and assumptions were applied to the modelling procedures:

- Modelling was based on the Trisobuild System (TATA Steel, 2011).
- Cladding assemblies were designed as double-span with 1.8m span (spanning from purlin to purlin).
- Fastener design resistances and flexibilities were according to Davies and Lawson (1999), which are considered conservative, and are shown in Table D.4. A comparison with the values derived based on BS 5950-9:1994 (BSI, 1994) is also given. It may be noted that the resistances in the British Standard are higher than those of quoted by Davies and Lawson (1999). Furthermore, the standard provides higher flexibilities for screws without plastic seal and lower flexibilities for those with plastic seal and monel blind rivets, compared to the aforementioned paper.
- It was assumed that plastic seal for fasteners will only be required for the weather-tight layer, i.e. the outer sheet.

- Liner sheet side-lap is not seam fastened in practice, rather sealed with tape. For the purposes of the analysis, seam-fastened liner side-laps were considered when estimating the composite action of the liner and outer sheeting. Seam fasteners spacing for the liner is considered equal to the spacing of the outer sheet seam fasteners, which is at 450mm.
- The case of the liner sheet side-laps not being fastened was also considered, and was treated as leaving the outer sheet alone to resist the in-plane forces.
- The liner sheet is fastened directly on to the purlin. Fastening is considered for both alternate corrugations (333.2mm pitch for LT1000, 400mm pitch for RL32) and every corrugation (166.6mm pitch for LT1000, 200mm pitch for RL32) as two separate cases.
- The outer sheet is fastened to the spacer bar component; however, since the spacer bar is in turn fastened to the purlin with the aid of the brackets, the model considered that the outer sheet is fastened to the purlin with fasteners as for the outer sheet – spacer bar interfaces. Similarly to the liner, fastening for both alternate (400mm pitch) and every (200mm pitch) corrugation(s) was considered as two separate cases.
- Shear connectors are required for the 4-sided diaphragms; in that case and since the use of shear connectors is not clear, it is assumed that they do not dominate the performance and, thus, ignored. For the 2-sided diaphragms, it is assumed that appropriate shear connectors are used at the gable rafters which are, again, ignored.
- Dismissing the influence of shear connectors is conservative for resistance purposes. In terms of flexibility, their contribution to the sum equation is equal to zero and, therefore, non-conservative.

Table D.4 Fastener resistances and flexibilities for built-up systems

Fastener	Envelope component & thickness	Resistance (kN) (Davies & Lawson, 1999)	Flexibility (mm/KN) (Davies & Lawson, 1999)	Resistance (kN) (BS5950-9: 1994)	Flexibility (mm/KN) (BS5950-9: 1994)
Sheet to purlin fasteners					
5.5mm dia. self-drilling, self-tapping screws with steel washers	Liner RL32 (0.7mm) Liner LT1000 (0.4mm)	4.6 x 0.7 = 3.22 3.8 x 0.4 = 1.52	0.06	5.45 x 0.7 = 3.81 5.45 x 0.4 = 2.18	0.15
5.5mm dia. self-drilling, self-tapping screws with steel washers and plastic seal	Outer R32 (0.7mm) (use of plastic seal for weather tightness)	4.1 x 0.7 = 2.87	0.45	4.36 x 0.7 = 3.05	0.35
Seam fasteners					
4.8mm dia. monel metal blind rivets	Outer R32 and liner RL32 (0.7mm) Liner LT1000 (0.4mm)	2.8 x 0.7 = 1.96 2.8 x 0.4 = 1.12	0.35	3.05 x 0.7 = 2.14 3.05 x 0.4 = 1.22	0.30

Note on K-factors calculations (as in BS 5950-9:1994): R32 / RL32: h/d=0.16, l/d=0.135, θ=35.4°; K1=0.055, K2=0.309; LT1000: h/d=0.114 l/d=0.15, θ=42.6°; K1=0.034, K2=0.197

D.1.2.2 Deck modelling details

The following input and assumptions were applied to the modelling procedures:

- Modelling was based on the Roofdeck D60 system (TATA Steel, 2012) and the Profil HV18 system (SkanDek, 2009).
- The assemblies were designed as double-span with 1.8m span (from purlin to purlin). Also, the Roofdeck D60 case is reviewed for multi-span conditions.
- Fastener design resistances and flexibilities were input based on the Davies and Lawson (1999) quantities, which are considered conservative, and are shown in Table D.5. A comparison with the values derived based on BS 5950-9:1994 is also given. It may be noted that the resistances in the British Standard are higher than those of quoted by Davies and Lawson (1999).
- It is assumed that no plastic seal for fasteners will be required since weather-tightness is achieved by layers on top of the decking.

- The decking is fastened directly on to the purlin. Fastening is considered for both alternate corrugations (400mm pitch for Roofdeck D60, 228mm pitch for Profil HV18) and every corrugation (200mm pitch for Roofdeck D60, 114mm pitch for Profil HV18) as two separate cases.
- Shear connectors are required for the 4-sided diaphragms; in that case and since the use of shear connectors is not clear, it is assumed that they do not dominate the performance and, thus, their strength and stiffness contribution is ignored. For the 2-sided diaphragms, it is assumed that appropriate shear fasteners are used at the gable rafters which are, again, ignored.
- Dismissing the influence of shear connectors is conservative for resistance purposes. In terms of flexibility, their contribution to the sum equation is equal to zero and, therefore, non-conservative.

Table D.5 Fastener resistances and flexibilities for decking systems

Fastener	**Envelope component & thickness**	**Resistance (kN)** (Davies & Lawson, 1999)	**Flexibility (mm/KN)** (Davies & Lawson, 1999)	**Resistance (kN)** (BS5950-9: 1994)	**Flexibility (mm/KN)** (BS5950-9: 1994)
Sheet to purlin fasteners					
5.5mm dia. self-drilling, self-tapping screws with steel washers	Deck (0.7mm)	4.6 x 0.7 = 3.22	0.06	5.45 x 0.7 = 3.81	0.15
Seam fasteners					
4.8mm dia. monel metal blind rivets	Deck (0.7mm)	2.8 x 0.7 = 1.96	0.35	3.05 x 0.7 = 2.14	0.30

Note on K-factors calculations (as in BS 5950-9:1994): Roofdeck D60: K1=0.175, K2=1.526; Profil HV18: K1=0.038, K2=0.285

D.2 Summary of design

The current section shows the output of the preliminary portal frame design for each of the adopted structural schemes for the small and medium 1-bay frame buildings. The results are shown in Table D.6 and Table D.7.

Table D.6 Frame member design (Small building, 1-bay frames)

Member	**Scheme**					
	1a		1b		2a	2b
	2-sided	4-sided	2-sided	4-sided	4-sided	4-sided
Rafter	UB 406x 178x54	UB 406x 178x54	UB 406x 178x54	**UB 356x 171x51**	**UB 457x 152x52**	**UB 356x 171x51**
Column	UB 457x 152x60	***UB 406x 178x60***	UB 457x 152x52	UB 457x 152x52	***UB 406x 178x60***	UB 457x 152x52
Purlins / Tie strut	202 Z 20	202 Z 20	202 Z 20	202 Z 20	CHS 48.3x2.9	CHS 48.3x2.9*
Eaves strut	CHS 88.9x3.2	CHS 88.9x3.2	CHS 88.9x3.2	CHS 88.9x3.2	CHS 88.9x3.2	CHS 88.9x3.2
Roof bracing	CHS 60.3x2.9	CHS 60.3x2.9	CHS 60.3x3.6	CHS 60.3x3.6	CHS 60.3x2.9	CHS 60.3x3.6
Wall bracing	CHS 101.6x3.2	CHS 101.6x3.2	CHS 101.6x3.2	CHS 101.6x3.2	CHS 101.6x3.2	CHS 101.6x3.2

Table D.7 Frame member design (Medium building, 1-bay frames)

Member	**Scheme**					
	1a		1b		2a	2b
	2-sided	4-sided	2-sided	4-sided	4-sided	4-sided
Rafter	UB 610x 229x125	UB 610x 229x125	UB 610x 229x113	**UB 610x 229x101**	UB 610x 229x125	**UB 610x 229x101**
Column	UB 762x 267x147	UB 762x 267x147	UB 762x 267x134	UB 762x 267x134	UB 762x 267x147	UB 762x 267x134
Purlins / Tie strut	172 Z 13	172 Z 13	172 Z 13	172 Z 13	CHS 76.1x2.9	CHS 76.1x2.9*
Eaves strut	CHS 168.3x5	CHS 168.3x5	CHS 168.3x5	CHS 168.3x5	CHS 168.3x5	CHS 168.3x5
Roof bracing	CHS 76.1x2.9	CHS 76.1x3.2	CHS 76.1x2.9	CHS 76.1x3.2	CHS 76.1x2.9	CHS 76.1x2.9
Wall bracing	CHS 139.7x5.6	CHS 139.7x5.6	CHS 139.7x5.6	CHS 139.7x5.6	CHS 139.7x5.6	CHS 139.7x5.6

Appendix E Optimised long span sandwich panels

E.1 PIR core data and derived mathematical relationships

The measured data for the PIR core mechanical properties are shown in Figure E.1 and Figure E.2.

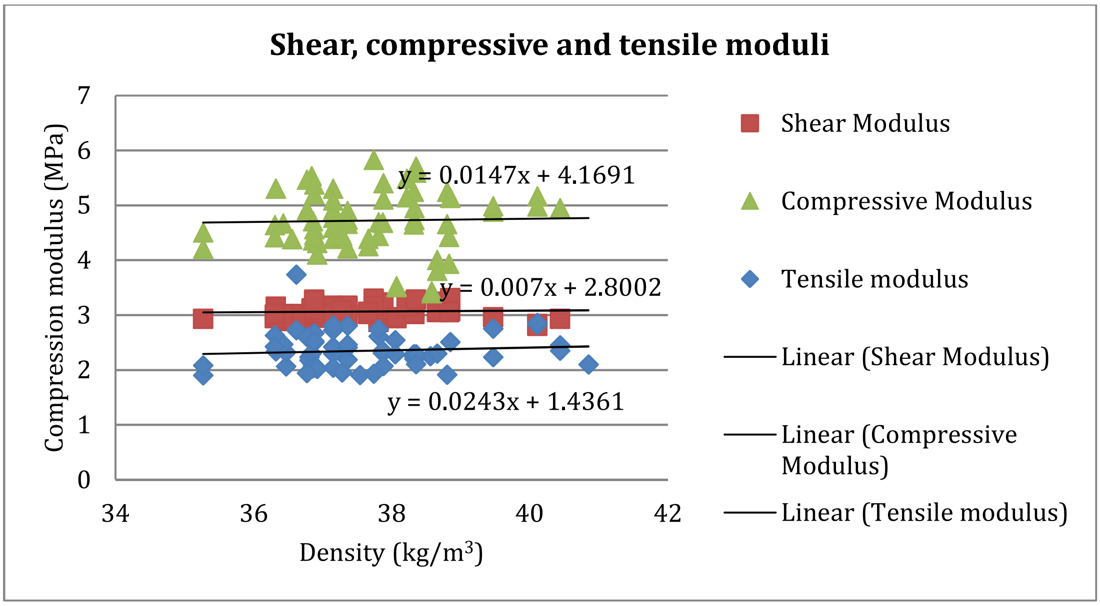

Figure E.1 Test results and linear relationships for PIR core shear, compressive and tensile moduli

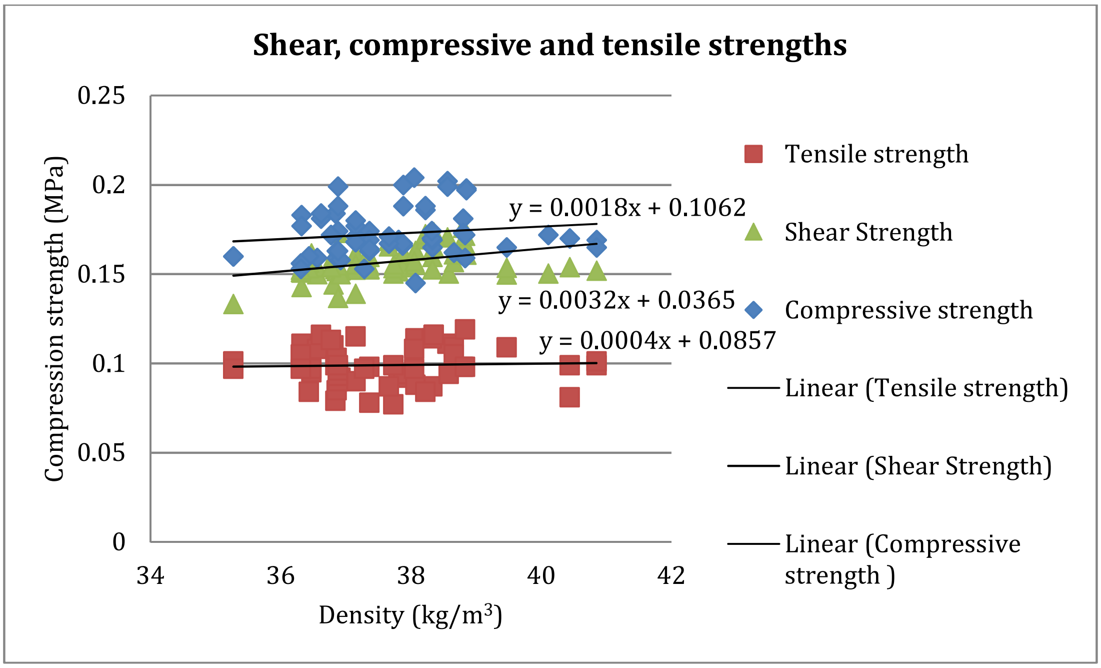

Figure E.2 Test results and linear relationships for PIR core shear, compressive and tensile strengths

The derived linear relationships are shown in Equation E.1 to Equation E.6.

$$E_{Cc} = 0.0147\rho_C + 4.1691$$ **Equation E.1**

$$E_{Ct} = 0.00243\rho_C + 1.4361$$ **Equation E.2**

$$G_C = 0.0070\rho_C + 2.8002$$ **Equation E.3**

$$f_{Cc} = 0.0018\rho_C + 0.1062$$ **Equation E.4**

$$f_{Ct} = 0.00048\rho_C + 0.0857$$ **Equation E.5**

$$f_{Cv} = 0.0032\rho_C + 0.0365$$ **Equation E.6**

The standard deviation for the sample is as shown in Table E.1. The fractile factor value was according to BS EN 14509:2013.

Table E.1 Standard deviation and fractile factor for tested core mechanical properties

Property	Standard deviation	Fractile factor
E_{Cc}	0.551	1.80*
E_{Ct}	0.543	
G_c	0.202	
f_{Cc}	0.022	
f_{Ct}	0.024	
f_{Cv}	0.016	

**For 88 test specimens according to BS EN 14509:2013*

The Poisson ratio test results are shown in Table E.4.

Table E.2 Poisson ratio test results

	Test 1	Test 2	Test 3
Density	31.6kg/m^3	33.0kg/m^3	31.9kg/m^3
ΔL (axial)	-3.71mm	-7.96mm	-10.44mm
ΔL (transverse)	0.34mm	0.84mm	0.79mm
Poisson ratio, v	0.09	0.11	0.08
Average	0.09		

E.2 Material properties of test specimens

Table E.3 Geometrical and material properties for PIR cores

Panel type	Panel thickness, d_C (mm)	Core density, ρ_C (kg/m^3)	Moduli – measured (mean)			Shear modulus local – calculated*
			G_c	E_{Cc}	E_{Ct}	G_C
A	120	40.1	2.51	2.06	1.18	0.79
B	120	36.0	3.20	4.85	2.25	1.63

**Using Poisson ratio v=0.09 (Section 7.3.2)*

Table E.4 Material properties for steel faces

Sheet type	Steel grade	Yield strength – measured (N/mm^2)	Ultimate strength – measured (N/mm^2)	Modulus of elasticity – nominal (N/mm^2)
A0	S220	357	416	210,000
A1	S220	372	436	
A2	S220			
B1	S220	319	367	
B2	S220			
B3	S220			
B4	S220			

E.3 Test apparatus

The load was applied by a pair of hydraulic jacks and distributed through square hollow section (SHS) spreader beams. 80mm x 80mm x 4mm SHS members were positioned beneath the longitudinal spreader beams. For the lightly profiled panels these were to ensure that the load was applied to the full panel width; for the fully profiled panels, timber blocks were used in between the longitudinal spreader beams and panel troughs. Rollers of 15mm radius were positioned between the longitudinal and transverse spreaders. The panels were supported on steel beams with a flange width of 150mm. A length of plywood was attached to the top flange of each beam to reduce the support width to 100mm as required by BS EN 14509:2013. The support conditions applied no rotational restraint to the panel about the line of support.

The tests were displacement controlled at a steady rate of 0.25mm/sec. The applied load was measured by calibrated load cells positioned below the two jacks. Displacements were recorded by transducers underneath the mid-span, the jacks and panel supports, as well as over the loading jacks for deflection control.

E.4 Test results

Table E.5 Test results: total load and stress at failure

Panel type	Sheet type	Test No.	Failure mode	Total load (kN)*	Stress at failure (N/mm²)	Mean stress (N/mm²)
A	A0	1	Local buckling	9.98	323.351	332.7
		2		10.41	340.742	
		3		10.65	341.852	
		4		9.94	322.233	
		5		10.30	335.396	
	A1	1	Local buckling and flexural wrinkling	10.31	195.165	210.3
		2		11.67	221.151	
		3		10.78	204.146	
		4		11.23	212.647	
		5		11.53	218.432	
	A2	1	Local buckling and flexural wrinkling	12.24	204.893	200.9
		2		11.52	192.890	
		3		12.01	201.051	
		4		12.75	213.378	
		5		11.49	192.342	
B	B1	1	Local buckling and flexural wrinkling	9.94	188.962	181.8
		2		9.23	174.633	
		3		9.88	187.742	
		4		10.34	197.251	
		5		8.54	160.446	
	B2	1	Local buckling and flexural wrinkling	8.94	143.631	163.2
		2		10.19	165.519	
		3		11.36	186.067	
		4		10.03	162.767	
		5		9.75	157.770	
	B3	1	Local buckling and flexural wrinkling	11.60	172.850	153.7
		2		11.49	171.050	
		3		11.22	166.793	
		4		11.31	168.125	
		5		10.02	147.579	
	B4	1	Local buckling and flexural wrinkling	15.99	146.735	165.3
		2		17.57	161.968	
		3		17.52	161.561	
		4		16.36	150.247	
		5		16.13	148.079	

**including panel self-weight.*

E.5 Optimisation variables

The variables shown in Table E.6 were kept constant and identical to the standard product.

Table E.6 Variables with constant values

Variable	Value	Note
p	333mm	Constant; for four full profiles per meter width.
$\boldsymbol{\varphi_R}$	126.3°	Constant; same as for current reference panel.
$\mathbf{d_{p1}}$	2.8mm	Constant; same as for current reference panel.
$\boldsymbol{\varphi_{p1}}$	58°	Constant; same as for current reference panel.
$\mathbf{d_{p2}}$	3mm	Constant; same as for current reference panel.
$\boldsymbol{\varphi_{p2}}$	30.5°	Constant; same as for current reference panel.

Appendix F Systems review

F.1 Embodied carbon coefficients

The following embodied carbon coefficients were used in the appraisal. The coefficients were based on a 'cradle-to-gate' approach and were extracted from the ICE Index from the University of Bath (Jones and Hammond, 2008).

Table F.1 Embodied carbon coefficients (Source: Jones and Hammond, 2008)

Component type / material	**Component type / material in the ICE Index**	**Embodied carbon emissions ($tnCO_2e/tn$)**
Fabricated steel sections (I-, H-)	Section	1.53
Steel hollow section	Pipe	1.45
Steel sheet (galvanised)	Coil (Sheet) – Galvanised	1.54
Cold-formed steel section	General	1.66
Steel reinforcement	Bar and rod	1.40
Concrete	General concrete 40/50	0.151
Aggregate	Aggregate	0.0052
Polyisocyanurate (PIR)	Polyurethane rigid foam	4.26
Mineral wool	Mineral wool	1.28
Polycarbonate	Polycarbonate	7.62
Glass – laminated	Toughened glass	1.35
Glass – toughened	Toughened glass	1.35

F.2 Envelope component weights

The following component weights for the envelope were used in the appraisal.

Table F.2 Component weights

Component	**Weight**
Flashing 200mm x 200mm x 0.7mm	2.2kg/m
Rooflights: Polycarbonate – U-value 1.3W/m^{2}K	3.6kg/m^2
Rooflights: GRP – U-value 1.4W/m^{2}K GRP	4.8kg/m^2
Northlights: Glass – U-value 1.2W/m^{2}K	3.4kg/m^2
Northlights: Polycarbonate - U-value 1.3W/m^{2}K	3.4kg/m^2
Sandwich panel roof cladding: Current – U-value 0.15W/m^{2}K (135mm - Appendix A)	
Total	12.34kg/m^2
Steel	6.99kg/m^2
PIR	5.35kg/m^2
Sandwich panel roof cladding: Enhanced – Optimised (Chapter 7)	
Total	13.77kg/m^2
Steel	7.52kg/m^2
PIR	6.25kg/m^2
Sandwich panel wall cladding: Current – U-value 0.17W/m^{2}K (120mm - Appendix A)	
Total	13.85kg/m^2
Steel	7.43kg/m^2
PIR	4.74kg/m^2
Built-up roof cladding: U-value 0.15W/m^{2}K	
Total	20.88kg/m^2
Steel	14.44kg/m^2
Mineral Wool	6.44kg/m^2

F.3 Cost rates

The following cost rates were used in the appraisal. The coefficients were extracted from the Spons' Handbook for Architects and Builders (AECOM, 2015).

Table F.3 Cost rates; Source: AECOM (2015)

Category	Activity	Component type	Unit	Cost
Structural metalwork	Material supply only	Universal beams and columns	tn	Basic price: £750 Added: £0-£80
		Hollow sections	tn	£866
	Framed members, framing, fabrication	Columns	tn	£1117-£1410
		Beams/Rafters	tn	£1086-£1205
		Trusses and built-up girders	tn	£2011.62
		Bracings	tn	£1334.0
		Purlins	m	£31.35-40.48
		Tubular ties	m	£17.19
		Cleats	No.	£15.86
Cladding and covering	Roof cladding – metal (manufacturing)	Sandwich panel – 135mm (current)	m^2	£50.37
		Sandwich panel – 150mm (enhanced)	m^2	£57.75
		Composite wall panel	m^2	£81.9
		Profiled sheeting 0.7mm R32	m^2	£16.59
		Profiled sheeting 0.5mm R32	m^2	£12.43
		Polycarbonate rooflight / northlight	m	£12.16
		GRP rooflights	m^2	£52.5
		Flashings	m^2	£3.15
	Roof – patent glazing	Doubled-glazed northlight	m^2	£130.41
Insulation, fire stopping and fire protection	Insulation	Mineral wool slabs, 90mm height	m^2	£5.67
In-situ concrete works	In-situ concrete	Reinforced C40 - Foundations	m^3	£108.67
		Reinforced C40 - Slabs	m^3	£111.17
Site clearance and preparation	Excavation	Excavating by machine to reduce levels	m^3	£2.08
	Filling obtained from excavated materials	Filling to make up levels	m^3	£4.27
	Geotextile	Geotextile fabric	m^2	£1.05

F.4 Mass, embodied carbon emissions and cost data

The breakdown of the systems in terms of mass is shown in Table F.4.

The breakdown of the systems in terms of embodied carbon is shown in Table F.5.

The breakdown of the systems in terms of cost is shown in Table F.6 to Table F.16.

Table F.4 Mass breakdown

Building size	Option	Frame spacing	Total	Frame	Frame	Purlins	Roof bracing	Wall bracing	Struts	Foundations	Ground floor slab - concrete	Ground floor slab - reinforcement	Sub-base filling
			tn	tn	tn	tn	tn	tn	tn	tn	tn	tn	tn
Small	1A	6.67m	36.9	17.8	13.8	2.1	0.9	0.4	0.5	85.8	573.4	4.5	500.0
Small	1B	6.67m	45.1	17.8	13.8	2.1	0.9	0.4	0.5	93.0	573.4	4.5	500.0
Small	2A/B	6.67m	35.5	11.9	8.8	0.0	1.5	0.4	1.1	84.6	573.4	4.5	500.0
Medium	1A	8m	132.7	64.1	51.3	8.6	1.8	0.7	1.7	1037.7	2293.6	18.0	2000.0
Medium	1B	8m	165.5	64.1	51.3	8.6	1.8	0.7	1.7	1128.0	2293.6	18.0	2000.0
Medium	2A	8m	112.3	28.4	23.5	0.0	3.0	1.1	0.8	981.6	2293.6	18.0	2000.0
Medium	2B	6.67m	111.0	32.0	26.8	0.0	2.5	0.7	2.0	1156.0	2293.6	18.0	2000.0
Large	1A	8.33m	374.1	226.1	190.8	25.5	2.8	0.9	6.0	3943.0	5733.9	45.0	5000.0
Large	1B	8.33m	455.8	226.1	190.8	25.5	2.8	0.9	6.0	4269.7	5733.9	45.0	5000.0
Large	2A	8.33m	308.9	114.1	104.7	0.0	4.7	1.1	3.5	3682.2	5733.9	45.0	5000.0
Large	2B	6.57m	297.2	112.0	117.7	0.0	3.9	0.8	3.5	4543.9	5733.9	45.0	5000.0

Building size	Option	Frame spacing	Roof cladding	Wall cladding	Rooflights	Flashings	Roof cladding	Wall cladding	Rooflights	Flashings
			m2	m2	m2	m	tn	tn	tn	tn
Small	1A	6.67m	854.7	552.8	150.8	186.3	10.5	7.6	0.5	0.4
Small	1B	6.67m	854.7	552.8	150.8	186.3	17.8	7.6	1.5	0.4
Small	2A/B	6.67m	1019.7	572.6	197.1	404.0	14.1	7.9	0.7	0.9
Medium	1A	8m	3419.0	1690.5	603.4	444.6	42.2	23.3	2.2	1.0
Medium	1B	8m	3419.0	1690.5	603.4	444.6	71.4	23.3	5.8	1.0
Medium	2A	8m	4053.6	1665.1	656.9	1197.0	56.1	22.9	2.2	2.6
Medium	2B	6.67m	4078.5	1664.9	786.0	1403.2	50.3	22.9	2.7	3.1

Building size	Option	Frame spacing	Roof cladding	Wall cladding	Rooflights	Flashings	Roof cladding	Wall cladding	Rooflights	Flashings
Large	1A	8.33m	8504.7	2628.0	1500.8	684.9	104.9	36.2	5.4	1.5
Large	1B	8.33m	8504.7	2628.0	1500.8	684.9	177.6	36.2	14.5	1.5
Large	2A	8.33m	10309.4	2722.7	2522.5	2731.0	142.8	37.5	8.6	6.0
Large	2B	6.57m	10485.1	2722.4	3195.2	3392.2	129.4	37.5	10.9	7.5

Building size	Option	Frame spacing	Steel-cladding	PIR-cladding	MW-cladding	Steel	Concrete	PIR	Mineral Wool	Polycarbonate / GRP	Glass	Aggregate
			tn	tn	tn	tn	tn	tn	tn	tn	tn	tn
Small	1A	6.67m	10.1	7.2		28.3	85.8	7.2		0.5		500.0
Small	1B	6.67m	16.4	2.6	5.5	34.7	93.0	2.6	5.5	0.5		500.0
Small	2A/B	6.67m	11.9	9.1		24.7	84.6	9.1		0.7	0.0	500.0
Medium	1A	8m	36.5	26.3		101.5	1037.7	26.3		2.2		2000.0
Medium	1B	8m	61.9	8.0	22.0	127.0	1128.0	8.0	22.0	2.2		2000.0
Medium	2A	8m	40.7	33.2		71.7	981.6	33.2		2.2	0.0	2000.0
Medium	2B	6.67m	54.7	27.2		89.8	1156.0	27.2		2.7	0.0	2000.0
Large	1A	8.33m	79.0	58.0		306.6	3943.0	58.0		5.4		5000.0
Large	1B	8.33m	142.3	12.5	54.8	369.9	4269.7	12.5	54.8	5.4		5000.0
Large	2A	8.33m	92.3	77.3		212.4	3682.2	77.3		8.6	0.0	5000.0
Large	2B	6.57m	118.2	62.6		237.6	4543.9	62.6		10.9	0.0	5000.0

Table F.5 Embodied carbon breakdown

Building size	Option	Frame spacing	Total	Frame total	Frame	Purlins	Roof bracing	Wall bracing	Struts	Foundations	Ground floor slab - concrete	Ground floor slab - reinforcement	Sub-base filling
			tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2
Small	1A	6.67m	91.2	27.4	21.1	3.6	1.3	0.6	0.8	13.0	86.6	6.3	2.6
Small	1B	6.67m	97.0	27.4	21.1	3.6	1.3	0.6	0.8	14.0	86.6	6.3	2.6
Small	2A/B	6.67m	89.6	18.0	13.5	0.0	2.2	0.6	1.6	12.8	86.6	6.3	2.6
Medium	1A	8m	441.8	98.8	78.5	14.2	2.6	1.0	2.5	156.7	346.3	25.2	10.4
Medium	1B	8m	474.0	98.8	78.5	14.2	2.6	1.0	2.5	170.3	346.3	25.2	10.4
Medium	2A	8m	419.9	43.0	36.0	0.0	4.3	1.6	1.1	148.2	346.3	25.2	10.4
Medium	2B	6.67m	437.8	48.6	41.0	0.0	3.6	1.1	2.9	174.5	346.3	25.2	10.4
Large	1A	8.33m	1355.9	348.5	291.9	42.4	4.1	1.4	8.6	595.4	865.8	63.0	26.0
Large	1B	8.33m	1451.5	348.5	291.9	42.4	4.1	1.4	8.6	644.7	865.8	63.0	26.0
Large	2A	8.33m	1284.4	173.8	160.3	0.0	6.8	1.6	5.1	556.0	865.8	63.0	26.0
Large	2B	6.57m	1410.3	192.0	180.1	0.0	5.7	1.1	5.1	686.1	865.8	63.0	26.0

Building size	Option	Frame spacing	Roof cladding	Wall cladding	Rooflights	Flashings
			tn eCO2	tn eCO2	tn eCO2	tn eCO2
Small	1A	6.67m	28.7	17.5	4.1	0.6
Small	1B	6.67m	26.1	17.5	11.4	0.6
Small	2A/B	6.67m	34.2	18.1	5.1	1.4
Medium	1A	8m	114.7	53.5	16.6	1.5
Medium	1B	8m	104.2	53.5	45.7	1.5

Building size	Option	Frame spacing	Roof cladding	Wall cladding	Rooflights	Flashings
Medium	2A	8m	154.9	52.7	17.0	4.1
Medium	2B	6.67m	136.9	52.7	20.4	4.7
Large	1A	8.33m	285.4	83.1	41.2	2.3
Large	1B	8.33m	259.2	83.1	113.6	2.3
Large	2A	8.33m	393.9	86.1	65.4	9.2
Large	2B	6.57m	351.8	86.1	82.8	11.5

Building size	Option	Frame spacing	Steel-cladding	PIR-cladding	MW-cladding	Steel	Concrete	PIR	Mineral Wool	Polycarbonate / GRP	Glass	Aggregate
			tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2	tn eCO2
Small	1A	6.67m	15.5	30.6		42.9	13.0	30.6		4.1	0.0	2.6
Small	1B	6.67m	25.3	11.2	7.0	52.7	14.0	11.2	7.0	4.1		2.6
Small	2A/B	6.67m	18.4	38.7		36.3	12.8	38.7		5.1	0.0	2.6
Medium	1A	8m	56.1	112.1		155.0	156.7	112.1		16.6	0.0	10.4
Medium	1B	8m	95.4	34.1	28.2	194.2	170.3	34.1	28.2	16.6		10.4
Medium	2A	8m	62.7	141.5		105.7	148.2	141.5			0.0	10.4
Medium	2B	6.67m	84.3	116.0		132.8	174.5	116.0			0.0	10.4
Large	1A	8.33m	121.6	246.9		470.1	595.4	246.9		41.2	0.0	26.0
Large	1B	8.33m	219.2	53.1	70.1	567.6	644.7	53.1	70.1	41.2		26.0
Large	2A	8.33m	142.1	329.5		315.9	556.0	329.5		65.4	0.0	26.0
Large	2B	6.57m	182.0	266.7		373.9	686.1	266.7			0.0	26.0

Table F.6 Small building – Option 1A cost breakdown

Component type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	14	tn	3.0	54	760	tn	2298.2	1233.5	tn	3730.1	212.1	tn	3726.2
Columns - Int	No.	0	tn	0.0	40	760	tn	0.0	1233.5	tn	0.0			
Rafters	No.	14	tn	10.6	60	760	tn	8024.0	1205.3	tn	12725.7			
Trusses - main	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Bracings - wall	No.	8	tn	1.3		866	tn	1130.2	1334.0	tn	1741.0			
Struts	No.	12	tn	0.5		866	tn	468.3	1344.0	tn	726.8			
Trusses - edge	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Base connections	No.	14												
Eaves connections	No.	14												
Apex connections	No.	7												
Foundations	No.	14	m3	33.7					108.7	m3	3659.0			
Purlins	No.	70	tn	2.1					7260.0	tn	15538.4			
Anti-sag rods	No.	78	m	150.8					17.2	m	2592.7			
Cleats	No.	98							15.9	part	1554.3			
Roof panels	No.	0	m2	854.7					50.4	m2	43054.3			
Wall panels	No.	48	m2	552.8					81.9	m2	45270.2			
Rooflights			m2	128.2					12.6	m2	1615.3			
Glazing			m2	0.0					12.6	m2	0.0			
Flashings			m	186.3					3.2	m2	586.8			

Table F.7 Small building – Option 1B cost breakdown

Component type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	14	tn	3.0	54	760	tn	2298.2	1233.5	tn	3730.1	212.1	tn	3726.2
Columns - Int	No.	0	tn	0.0	40	760	tn	0.0	1233.5	tn	0.0			
Rafters	No.	14	tn	10.6	60	760	tn	8024.0	1205.3	tn	12725.7			
Trusses	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Bracings	No.	8	tn	1.3		866	tn	1130.2	1334.0	tn	1741.0			
Struts	No.	12	tn	0.5		866	tn	468.3	1344.0	tn	726.8			
Trusses - edge	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Base connections	No.	14												
Eaves connections	No.	14												
Apex connections	No.	7												
Foundation pads	No.	14	m3	36.5					108.7	m3	3965.3			
Purlins	No.	70	tn	2.1					7260.0	tn	15538.4			
Anti-sag rods	No.	78	m	150.8					17.2	m	2592.7			
Cleats	No.	98							15.9	part	1554.3			
Roof panels	No.	0	m2	854.7					46.0	m2	39340.7			
Wall panels	No.		m2	552.8					81.9	m2	45270.2			
Rooflights			m2	128.2					52.5	m2	6730.6			
Glazing			m2	0.0					12.6	m2	0.0			
Flashings			m	186.3					3.2	m	586.8			

Table F.8 Small building – Option 2A/B cost breakdown

Component type	**Unit**	**Quantity**	**Unit**	**Quantity**	**Weight**	**Material**			**Fabrication**			**Erection**		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	14	tn	1.1	19	760	tn	808.6	1410.8	tn	1501.1	212.1	tn	2527.5
Columns - Int	No.	0	tn	0.0	40	760	tn	0.0	1233.5	tn	0.0			
Rafters	No.	0	tn	0.0		760	tn	0.0	1205.3	tn	0.0			
Trusses	No.	14	tn	7.8		866	tn	6741.0	2011.6	tn	15658.5			
Bracings	No.	8	tn	2.0		866	tn	1695.1	1334.0	tn	2611.1			
Struts	No.	0	tn			866	tn	0.0	2011.6	tn	0.0			
Trusses - edge	No.	12	tn	1.1		866	tn	962.2	2011.6	tn	2235.0			
Base connections	No.	14												
Eaves connections	No.	14												
Apex connections	No.	7												
Foundation pads	No.	14	m3	33.2					108.7	m3	3606.0			
Purlins	No.	0	tn	0.0					7260.0	tn	0.0			
Anti-sag rods	No.	0	m	0.0					17.2	m	0.0			
Cleats	No.	0							15.9	part	0.0			
Roof panels	No.	153	m2	1019.7					50.4	m2	51368.6			
Wall panels	No.		m2	572.6					81.9	m2	46894.2			
Rooflights			m2	0.0					12.6	m2	0.0			
Glazing			m2	197.1					12.6	m2	2483.1			
Flashings			m	404.0					3.2	m	1272.5			

Table F.9 Medium building – Option 1A cost breakdown

Type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	22	tn	7.9	60	760	tn	6019.2	1233.5	tn	9769.4	212.1	tn	20667.1
Columns - Int	No.	11	tn	2.6	40	760	tn	2006.4	1233.5	tn	3256.5			
Rafters	No.	44	tn	74.1	67	760	tn	56320.5	1205.3	tn	89322.1			
Trusses - main	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Bracings	No.	8	tn	2.5		866	tn	2156.0	1334.0	tn	3321.1			
Struts	No.	20	tn	1.7		866	tn	1496.4	1344.0	tn	2322.4			
Trusses - edge	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Base connections	No.	22												
Eaves connections	No.	22												
Apex connections	No.	22												
Foundation pads	No.	22	m3	305.4					108.7	m3	33186.7			
Purlins	No.	196	tn	8.6					7260.0	tn	62122.4			
Anti-sag rods	No.	540	m	754.1					17.2	m	12963.5			
Cleats	No.	308							15.9	part	4884.9			
Roof panels	No.	0	m2	3419.0					50.4	m2	172234.1			
Wall panels	No.		m2	1690.5					81.9	m2	138452.0			
Rooflights			m2	512.9					12.6	m2	6462.0			
Glazing			m2	0.0					12.6	m2	0.0			
Flashings			m	444.6					3.2	m2	1400.3			

Table F.10 Medium building – Option 1B cost breakdown

Type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	22	tn	7.9	60	760	tn	6019.2	1233.5	tn	9769.4	212.1	tn	20667.1
Columns - Int	No.	11	tn	2.6	40	760	tn	2006.4	1233.5	tn	3256.5			
Rafters	No.	44	tn	74.1	67	760	tn	56320.5	1205.3	tn	89322.1			
Trusses - main	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Bracings	No.	8	tn	2.5		866	tn	2156.0	1334.0	tn	3321.1			
Struts	No.	20	tn	1.7		866	tn	1496.4	1344.0	tn	2322.4			
Trusses - edge	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Base connections	No.	22												
Eaves connections	No.	22												
Apex connections	No.	22												
Foundation pads	No.	22	m3	332.0					108.7	m3	36074.7			
Purlins	No.	196	tn	8.6					7260.0	tn	62122.4			
Anti-sag rods	No.	540	m	754.1					17.2	m	12963.5			
Cleats	No.	308							15.9	part	4884.9			
Roof panels	No.	0	m2	3419.0					46.0	m2	157378.4			
Wall panels	No.		m2	1690.5					81.9	m2	138452.0			
Rooflights			m2	512.9					52.5	m2	26924.9			
Glazing			m2	0.0					12.6	m2	0.0			
Flashings			m	444.6					3.2	m2	1400.3			

Table F.11 Medium building – Option 2A cost breakdown

Type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	22	tn	3.3	25	750	tn	2475.0	1410.8	tn	4655.5	212.1	tn	6553.8
Columns - Int	No.	11	tn	3.0	45	760	tn	2257.2	1233.5	tn	3663.5			
Rafters	No.	0	tn	0.0			tn	0.0	1205.3	tn	0.0			
Trusses - main	No.	33	tn	19.8		866	tn	17143.3	2011.6	tn	39822.0			
Bracings	No.	8	tn	4.1		866	tn	3525.7	1334.0	tn	5431.1			
Struts	No.	0	tn			866	tn	0.0	2011.6	tn	0.0			
Trusses - edge	No.	20	tn	0.8		866	tn	660.2	2011.6	tn	1533.5			
Base connections	No.	22												
Eaves connections	No.	22												
Apex connections	No.	22												
Foundation pads	No.	22	m3	288.9					108.7	m3	31394.7			
Purlins	No.	0	tn	0.0					7260.0	tn	0.0			
Anti-sag rods	No.	0	m	0.0					17.2	m	0.0			
Cleats	No.	0							15.9	part	0.0			
Roof panels	No.	507	m2	4053.6					57.8	m2	234094.3			
Wall panels	No.		m2	1665.1					81.9	m2	136372.0			
Rooflights			m2	0.0					12.6	m2	0.0			
Glazing			m2	656.9					12.6	m2	8277.0			
Flashings			m	1197.0					3.2	m	3770.7			

Table F.12 Medium building – Option 2B cost breakdown

Type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	26	tn	3.9	25	750	tn	2925.0	1410.8	tn	5502.0	212.1	tn	6178.5
Columns - Int	No.	13	tn	3.1	40	760	tn	2371.2	1233.5	tn	3848.6			
Rafters	No.	0	tn	0.0			tn	0.0	1205.3	tn	0.0			
Trusses - main	No.	39	tn	17.3		866	tn	14961.4	2011.6	tn	34753.8			
Bracings	No.	8	tn	4.1		866	tn	3525.7	1334.0	tn	5431.1			
Struts	No.	0	tn			866	tn	0.0	2011.6	tn	0.0			
Trusses - edge	No.	24	tn	0.8		866	tn	660.2	2011.6	tn	1533.5			
Base connections	No.	26												
Eaves connections	No.	26												
Apex connections	No.	26												
Foundation pads	No.	26	m3	340.2					108.7	m3	36969.4			
Purlins	No.	0	tn	0.0					7260.0	tn	0.0			
Anti-sag rods	No.	0	m	0.0					17.2	m	0.0			
Cleats	No.	0							15.9	part	0.0			
Roof panels	No.	612	m2	4078.5					50.4	m2	205452.2			
Wall panels	No.		m2	1664.9					81.9	m2	136351.4			
Rooflights			m2	0.0					12.6	m2	0.0			
Glazing			m2	786.0					12.6	m2	9903.6			
Flashings			m	1403.2					3.2	m	4420.1			

Table F.13 Large building – Option 1A cost breakdown

Type	**Unit**	**Quantity**	**Unit**	**Quantity**	**Weight**	**Material**			**Fabrication**			**Erection**		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	32	tn	25.7	134	780	tn	20067.8	1233.5	tn	31735.7	212.1	tn	75897.8
Columns - Int	No.	16	tn	5.2	54	830	tn	4302.7	1117.2	tn	5791.4			
Rafters	No.	64	tn	290.9	113	760	tn	221063.8	1086.3	tn	315964.2			
Trusses - main	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Bracings	No.	8	tn	8.8		866	tn	7615.6	1334.0	tn	11731.3			
Struts	No.	30	tn	1.7		866	tn	1496.4	1344.0	tn	2322.4			
Trusses - edge	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Base connections	No.	32												
Eaves connections	No.	32												
Apex connections	No.	32												
Foundation pads	No.	32	m3	1160.4					108.7	m3	126103.7			
Purlins	No.	437	tn	25.5					7260.0	tn	185362.6			
Anti-sag rods	No.	1350	m	1809.9					17.2	m	31112.4			
Cleats	No.	736							15.9	part	11673.0			
Roof panels	No.	0	m2	8504.7					50.4	m2	428423.0			
Wall panels	No.		m2	2628.0					81.9	m2	215233.2			
Rooflights			m2	1275.7					12.6	m2	16073.8			
Glazing			m2	0.0					12.6	m2	0.0			
Flashings			m	684.9					3.2	m2	2157.4			

Table F.14 Large building – Option 1B cost breakdown

Type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	32	tn	25.7	134	780	tn	20067.8	1233.5	tn	31735.7	212.1	tn	75897.8
Columns - Int	No.	16	tn	5.2	54	830	tn	4302.7	1117.2	tn	5791.4			
Rafters	No.	64	tn	290.9	113	760	tn	221063.8	1086.3	tn	315964.2			
Trusses - main	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Bracings	No.	8	tn	8.8		866	tn	7615.6	1334.0	tn	11731.3			
Struts	No.	30	tn	1.7		866	tn	1496.4	1344.0	tn	2322.4			
Trusses - edge	No.	0	tn	0.0		866	tn	0.0	2011.6	tn	0.0			
Base connections	No.	32												
Eaves connections	No.	32												
Apex connections	No.	32												
Foundation pads	No.	32	m3	1256.6					108.7	m3	136552.7			
Purlins	No.	437	tn	25.5					7260.0	tn	185362.6			
Anti-sag rods	No.	1350	m	1809.9					17.2	m	31112.4			
Cleats	No.	736							15.9	part	11673.0			
Roof panels	No.	0	m2	8504.7					46.0	m2	391470.2			
Wall panels	No.		m2	2628.0					81.9	m2	215233.2			
Rooflights			m2	1275.7					52.5	m2	66974.3			
Glazing			m2	0.0					12.6	m2	0.0			
Flashings			m	684.9					3.2	m2	2157.4			

Table F.15 Large building – Option 2A cost breakdown

Type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	32	tn	5.8	30	750	tn	4320.0	1410.8	tn	8126.0	212.1	tn	24195.4
Columns - Int	No.	16	tn	5.8	60	760	tn	4377.6	1233.5	tn	7105.0			
Rafters	No.	0	tn	0.0			tn	0.0	1205.3	tn	0.0			
Trusses - main	No.	48	tn	93.2		866	tn	80736.1	2011.6	tn	187540.9			
Bracings	No.	8	tn	5.8		866	tn	5017.1	1334.0	tn	7728.5			
Struts	No.	0	tn			866	tn	0.0	2011.6	tn	0.0			
Trusses - edge	No.	30	tn	3.5		866	tn	3059.7	2011.6	tn	7107.2			
Base connections	No.	32												
Eaves connections	No.	32												
Apex connections	No.	32												
Foundation pads	No.	32	m3	1083.7					108.7	m3	117762.7			
Purlins	No.	0	tn	0.0					7260.0	tn	0.0			
Anti-sag rods	No.	0	m	0.0					17.2	m	0.0			
Cleats	No.	0							15.9	part	0.0			
Roof panels	No.	1238	m2	10309.4					57.8	m2	595365.9			
Wall panels	No.		m2	2722.7					81.9	m2	222985.5			
Rooflights			m2	0.0					12.6	m2	0.0			
Glazing			m2	2522.5					12.6	m2	31783.5			
Flashings			m	2731.0					3.2	m	8602.7			

Table F.16 Large building – Option 2B cost breakdown

Type	Unit	Quantity	Unit	Quantity	Weight	Material			Fabrication			Erection		
					kg/m	Price	Unit	Price total	Price	Unit	Price total	Price	Unit	Price total
Columns - Ext	No.	40	tn	6.0	25	750	tn	4500.0	1410.8	tn	8464.6	212.1	tn	26704.1
Columns - Int	No.	20	tn	6.1	51	760	tn	4651.2	1233.5	tn	7549.1			
Rafters	No.	0	tn	0.0			tn	0.0	1205.3	tn	0.0			
Trusses - main	No.	60	tn	105.6		866	tn	91465.0	2011.6	tn	212462.8			
Bracings	No.	8	tn	4.7		866	tn	4039.0	1334.0	tn	6221.8			
Struts	No.	0	tn			866	tn	0.0	2011.6	tn	0.0			
Trusses - edge	No.	38	tn	3.5		866	tn	3032.3	2011.6	tn	7043.6			
Base connections	No.	40												
Eaves connections	No.	40												
Apex connections	No.	40												
Foundation pads	No.	40	m3	1337.3					108.7	m3	145321.1			
Purlins	No.	0	tn	0.0					7260.0	tn	0.0			
Anti-sag rods	No.	0	m	0.0					17.2	m	0.0			
Cleats	No.	0							15.9	part	0.0			
Roof panels	No.	1596	m2	10485.1					50.4	m2	528186.8			
Wall panels	No.		m2	2722.4					81.9	m2	222964.8			
Rooflights			m2	0.0					12.6	m2	0.0			
Glazing			m2	3195.2					12.6	m2	40259.1			
Flashings			m	3392.2					3.2	m	10685.5			